STRUCTURAL DETAILS

OR

ELEMENTS OF DESIGN

IN

HEAVY FRAMING

BY

HENRY S. JACOBY

PROFESSOR OF BRIDGE ENGINEERING IN CORNELL UNIVERSITY

FIRST EDITION

FIRST THOUSAND

NEW YORK

JOHN WILEY & SONS

LONDON: CHAPMAN & HALL, LIMITED

1909

First edition, September, 1909.

Set up and electrotyped by the J. S. Cushing Co., Norwood, Mass.
Printed and bound by Braunworth & Co., Brooklyn, N.Y.

PREFACE.

The title of this volume corresponds to a course of instruction conducted by the author in the College of Civil Engineering in Cornell University during the past nineteen years. In this course the students receive their first instruction in the application of the principles of mechanics to the design of the details of structures. Experience has shown that in many respects problems involving timber construction are better adapted for this purpose than if confined to structural steel.

It may appear at first as if too much attention to details is given in the examples on the design of joints, beams, and trusses. It is believed, however, that the importance of careful study of every detail can only thus be properly emphasized. In practice it seems to be the exception rather than the rule to give the same attention to details of timber structures as to those of steel. In the interest of sound engineering practice it is essential that all connections and details have the same degree of security as the framed members.

In several articles the order of design is given in full, with a view of economizing the time of the student, and of promoting systematic habits in making the computations required, these objects being regarded as important elements in efficient engineering education and practice.

Chapter I contains the digested results of extensive experimental investigation and research. Since their theoretic and practical value depend largely upon the conditions under which the experiments are made, and the limitations of space preclude their complete description, the original sources of information are given in every case.

A subject embracing so many details of design and construction cannot be treated fully in a single volume of convenient size, since it is necessary to observe its application in several different cases to obtain an adequate idea of the value or use of any detail, whether it be a fastening or a joint. The engineering periodicals and transactions contain a large amount of such illustrative material, and hence numerous but carefully selected references to them are made a prominent feature of this work. Besides directing the student in his reading on the topics included, they encourage him to form the habit of consulting these great cyclopædias of engineering practice.

To facilitate the use of this volume as a text-book, it has been so arranged that computations for the design of joints given in Chapter II may be commenced at the beginning of the course of instruction. In preparing the more strictly theoretic portion of the text a knowledge of mechanics has been assumed throughout. An attempt has been made to present the subjects of joints and fastenings in a systematic manner, and to include practical information on timber construction.

Grateful acknowledgments for photographs are due to Ralph Modjeski, H. H. Quimby, F. W. Skinner, and J. A. Knighton; for drawings to Henry Goldmark, J. S. Sewell, W. J. Douglas, and T. G. Pihlfeldt; to H. E. Stevens, C. L. Todd, and R. M. Bowman for permission to reproduce drawings; to Engineering News, Engineering Record, and Railroad Age Gazette for permission to reprint illustrations; to Hurlbut S. Jacoby for assistance in computation and drawing; and to J. P. Snow, George M. Heller, James Keys, W. M. Torrance, J. P. Whiskeman, and many other engineers, who have kindly furnished information.

SEPTEMBER 9, 1909.

CONTENTS.

Chapter I.

FASTENINGS USED IN FRAMING.

Chapter II.

JOINTS USED IN FRAMING.

CHAPTER III.

WOODEN BEAMS AND COLUMNS.

CHAPTER IV.

WOODEN ROOF TRUSSES.

CHAPTER V.

EXAMPLES OF FRAMING IN PRACTICE.

CHAPTER VI.

TIMBER TESTS AND UNIT-STRESSES.

FOLDING PLATES.

FULL-PAGE ILLUSTRATIONS.

STRUCTURAL DETAILS.

CHAPTER I.

FASTENINGS USED IN FRAMING.

ART. 1. BOLTS AND NUTS.

A bolt consists of a rod with suitable attachments at its ends which is used to hold together two or more pieces in any structure. The kind of bolt most frequently employed is shown in Fig. 1*a*, and has a permanent head at one end, while the other end is threaded to receive a nut. It is called a screw-bolt when it is necessary to distinguish it from other kinds of bolts.

Bolts are classified according to the shape of the head, or some structural feature of the head, by the mode in which they are secured or held in place; or by the purpose for which they are employed. The terms used to designate the numerous kinds of bolts may be found by reference to the word 'bolt' in the large dictionaries.

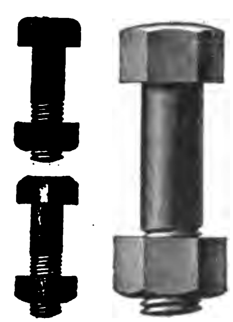


FIG. 1*a*. Bolts.

Bolts are manufactured either as rough or finished. The former are commonly used for woodwork. Finished bolts are obtained by turning rough bolts to exact dimensions, the holes being also drilled to an exact fit. They are used in bolting metal parts as, for example, in machinery.

The standard form and dimensions of screw threads in general use in this country were devised by WILLIAM SELLERS and recommended by the Franklin Institute in December, 1864. See

Journal of the Franklin Institute, third series, vol. 47, page 344, and vol. 49, page 53. The proportions recommended for nuts and bolt heads, however, have not been generally adopted because of the odd size of bar required to make them. The proportions in use are known as the manufacturers' standard. Tables of dimensions of standard screw threads, nuts, and bolt heads, are given in the manufacturers' handbooks, some of which, however, are much more convenient than others.

The thickness of a rough bolt head is made sufficient to develop the safe net tensile strength of the bolt, but the nut is made thicker, or equal to the diameter of the bolt, probably to provide extra wearing surface for the thread. The finished head and nut are made the same thickness to improve the appearance of finished work, as they frequently alternate in position or come into close proximity.

Under certain circumstances the thickness of bolt heads and nuts may vary from the standards. For instance, a rough bolt head for soft timber is often made large and thin, while a clamping nut, which is frequently screwed tight and unscrewed again, is made much thicker than the rule indicates for permanent fastenings.

With respect to the principal stresses in the body of the bolt, a bolt may be used either to prevent the separation of two or more parts of a structure in the direction of the bolts, or to prevent them from sliding past each other. In the former case the body of the bolt is subject to tension and in the latter case to compression and shear, either with or without flexure. When a bolt connects timbers which tend to slide on the surface of contact, the shear in any cross section need not be considered, as the bolt will always bend more or less on account of the yielding of the wooden fibers near the plane of contact. In this case the hole should be bored so as to secure a close or driving fit. For this purpose in bolting the longitudinal and sway braces to the

bents of wooden trestle bridges, it is customary to use an auger $\frac{1}{8}$ inch smaller in diameter than the bolt for bolts having a diameter of $\frac{3}{4}$ inch or over, and $\frac{1}{8}$ inch smaller for bolts with a diameter of $\frac{5}{8}$ inch or less. For the description and illustration of an ingenious device designed to increase the resistance of a bolted joint to shear in the plane of contact see an article on The Use of a Double-cone Washer for Timber Joints, by L.S. AUSTIN, in Engineering News, vol. 52, page 348, Oct. 20, 1904.

The length of a bolt is the distance from the inside of the head to the end of the screw. The grip of a bolt is the distance between the inside faces of the washers, or the total thickness of the material to be held together.

The design of a bolt subject to tension only requires the computation of the net section area at the root of the thread and the selection of a diameter of bolt from a table of commercial sizes which provides a net area equal to or slightly exceeding the one required. If a bolt has to be screwed up when the maximum direct tension is acting, it is necessary to increase the net section to provide for the additional stresses due to torsion. This increase may be taken at approximately 15 percent.

Experiments made by W. R. KING show that for a $1\frac{1}{2}$-inch wrought-iron bolt 12 and 18 threads to the inch give respectively 21 and 23 percent greater strength than the standard number of 6 threads, and a corresponding increase of elongation of 140 and 220 percent. These results indicate that for some special purposes where a large number of bolts are required, it may be desirable to increase the number of threads per inch, probably not to exceed double the standard number. See Report of the Chief of Engineers, U. S. A., 1885, page 1759, and also a paper by CHARLES T. PORTER in Transactions American Society of Mechanical Engineers, 1903, vol. 24, page 137.

In long bolts there is considerable waste of metal owing to the excess of section of the body of the bolt over that at the root of

the thread at its ends. This is avoided by using what are technically known as upset screw ends. Their diameter is enlarged so that their strength is at least equal to that of the body of the bolt. To secure this result the area at the root of the thread must be somewhat larger than that of the main rod, since the unit tensile strength of the material is reduced by the process of upsetting. Rods having a diameter of 1 inch should be upset for lengths of 12 feet and over, while 2½-inch rods should be upset for lengths of 6 feet and over. For intermediate sizes the lengths may be varied between the limits given. For shorter lengths it is more economical to use rods without upset ends.

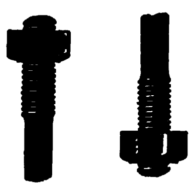


FIG. 1*b*. Upset Screw Ends.

FIG. 1*c*. Bolt with cold-pressed threads.

Tables of upset screw ends for both round and square bars are given in the manufacturers' handbooks, their proportions based on numerous tests of finished bars. To reduce the number of forms or dies required, the diameter of the upset varies by eighths of an inch, while that of the rods varies by sixteenths of an inch. For some data and discussion on the strength of upset and plain screw ends see Engineering Record, 1895, vol. 32, pages 83, 102, 120 and 257, and Railroad Age Gazette, vol. 45, page 611, July 31, 1908.

Instead of cutting the threads in a lathe, a process of cold rolling has been invented in which the threads and shoulders are made by depressing and shaping the fibers so as to retain their tensile strength. In another process the thread is raised slightly above the surface of the shank or body of the bolt by cold pressure, thus preserving the full strength of the body in the threaded portion (Fig. 1*c*). Some bolts are made with a helicoid shank which prevents the bolt from turning when the nut is screwed in place.

When there is danger of a nut working loose under the influence of vibration or other causes, various devices are provided to lock the nut. One of these consists in driving a nail alongside into the timber through a slot in the washer (see Fig. *2e*) provided for this purpose. A check or jam nut may be placed on top of the standard nut and brought into a firm bearing with it.

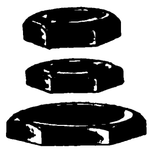


FIG. *1d*. Check Nuts. FIG. *1e*. Elastic Nut.

Check nuts have the same diameter, but only a trifle more than half of the thickness, of the standard form. Still another device consists of a self-locking nut having a split side and tapped slightly smaller than the standard, so that when wrenched on the bolt the side opens a trifle, but the bolt is held with sufficient grip to prevent working loose. In case the nut is not intended to be unscrewed at any time after the bolt is put in place, it may be locked by deforming the adjacent outer threads with a center punch.

Prob. 1. Find the diameter of a bolt required to resist 18 500 pounds in tension, the working unit stress being specified as 16 000 pounds per square inch.

ART. 2. WASHERS.

A washer is a perforated disk of iron or steel placed under the nut or head of a bolt. When the bolt passes through timber, the object of the washer is usually to provide sufficient bearing area so that the compression on the surface of the timber may not exceed a safe value. Sometimes a small washer may be used merely to avoid defacing the surface.

Washers are made of cast iron, malleable iron, wrought iron, or steel. The common form of cast-iron washer is round with a

cross section like that shown in Fig. *2a*, and is known as the 'ogee washer.' On drawings it is often designated as 'O.G. washer.' For the Pittsburgh standards given in the accompany-

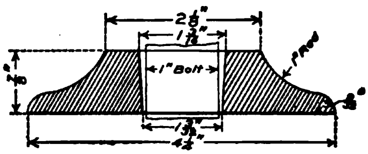


FIG. *2a*. Section of Ogee Washer.

ing table, the size of the hole at the bottom or back is $\frac{1}{8}$ inch larger than that of the bolt, and the size of the hole at the top or face is $\frac{1}{8}$ inch larger than at the back for the smaller sizes, and $\frac{1}{4}$ inch for the larger sizes. The diameter of the top or face of the washer is $(2\,d + 0.5)$ inches, d being the diameter of the bolt.

CAST-IRON OGEE WASHERS.

DIAM. OF BOLT.	DIAM. OF WASHER.	THICKNESS.	WEIGHT OF 100.	BEARING AREA	DIAM. OF BOLT.	DIAM. OF WASHER	THICKNESS.	WEIGHT OF 100.	BEARING AREA.
Inches.	Inches.	Inches.	Pounds.	Sq. In.	Inches.	Inches.	Inches.	Pounds.	Sq. In.
3/8	1 1/2	5/16	8.5	1.57	1	4	7/8	180	11.57
1/2	2	3/8	22	2.83	1	4	1	194	11.57
1/2	2 3/8	1/2	37.5	4.12	1 1/8	4 1/2	1	215	14.68
5/8	2 1/2	1/2	45	4.47	1 1/8	4 1/2	1 1/8	295	14.68
5/8	3	5/8	75	6.63	1 1/4	5	1 1/8	320	18.15
3/4	3	5/8	72	6.47	1 1/4	5 3/8	1 1/4	469	21.21
3/4	3 1/4	3/4	100	7.69	1 3/8	5 1/2	1 1/4	425	21.99
7/8	3 1/2	3/4	115	8.84	1 1/2	6	1 3/8	525	26.20
7/8	3 5/8	7/8	150	9.54	1 1/2	6	1 1/2	688	26.20

The proportions of cast washers adopted by some of the manufacturers differ somewhat in details. For example, the diameter of the face is sometimes made $(2\,d + 0.125)$ inches. Referring to Fig. *2b*, the radius F for a $\frac{3}{4}$ inch bolt is $\frac{1}{2}$ inch, in one case and $\frac{3}{4}$ inch in another, while the corresponding values of the radius G are $\frac{5}{16}$ and $\frac{7}{32}$ inch. In the form illus-

trated in Fig. $2a$ the radius of the upper curve equals the diameter of the bolt, while that of the lower curve is approximately one-fourth as large.

Most of the cast washers supplied by the trade have an outside diameter approximately equal to four times the diameter of the bolt, although the bearing area when used on soft wood

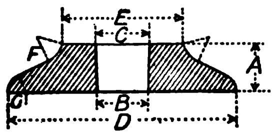


FIG. 2b.
Section of Standard Ogee Washer.

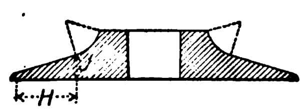


FIG. 2c.
Section of Special Ogee Washer.

should be larger than when used on hard wood. Some of the soft woods like white or Norway pine require three times as large a bearing area under washers as hard woods like white oak. The thickness is made either equal to the diameter of the bolt or $\frac{1}{8}$ inch less.

When no standard washer can be obtained that supplies sufficient bearing area, it becomes necessary to design special sizes. The accompanying table gives the proportions of those

PROPORTIONS OF SPECIAL OGEE WASHERS.

Bolt	A	B	C	D	E	F	G	H	J
1	1	$1\frac{1}{8}$	$1\frac{1}{4}$	6	$2\frac{1}{4}$	$\frac{3}{4}$	$\frac{3}{16}$	$1\frac{1}{8}$	$\frac{1}{2}$
$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	9	$3\frac{1}{2}$	$1\frac{1}{8}$	$\frac{1}{4}$	$1\frac{5}{8}$	$\frac{5}{8}$
2	2	$2\frac{1}{8}$	$2\frac{1}{4}$	12	$4\frac{1}{2}$	$1\frac{7}{16}$	$\frac{1}{4}$	$2\frac{1}{2}$	1

designed in 1888 for the wooden cribs and caissons of the Merchants' bridge at St. Louis. The diameters of the washers are observed to equal six times the diameters of the corresponding bolts. The weight of the smallest one is 3 pounds. Fig. $2d$ shows a countersunk washer used on the same foundation structures. The end of the bolt is also rounded to offer as little resistance as possible to any obstruction.

Fig. 2*f* shows the form of an unusually large cast-iron washer for a 2-inch rod used in repairing some masonry foundation piers of a building. The metal is 1½ inches thick throughout,

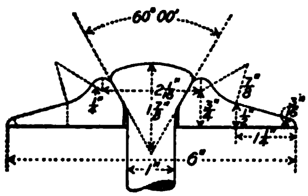


FIG. 2*d*. Section of Special Countersunk washer.

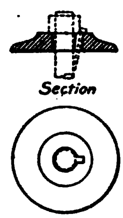


FIG. 2*e*. Slotted Washer.

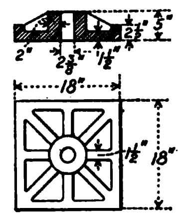
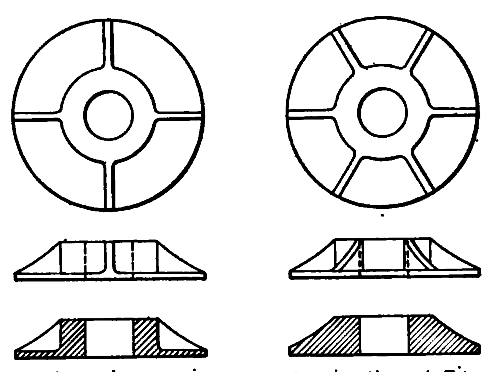


FIG. 2*f*. Washer for Bearing on Masonry.

except that in the central cylinder or spool which is 2 inches in thickness. See Engineering Record, vol. 35, page 29, Dec. 12, 1896. Fig. 2*e* shows a slotted washer which differs from the ordinary form by the addition of a slot through which a nail may be driven for the purpose of locking the nut in its final position. Cast washers are also made diamond shaped, the dimensions of which may be found in the manufacturers' catalogues.

Fig. 2*h* illustrates the general form of a cast-iron washer designed in 1903 by A. F. ROBINSON for use in the construction



FIGS. 2*g* and *h*. Special Designs of Cast Washers.

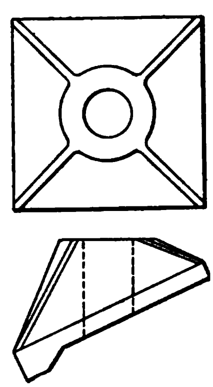


FIG. 2*i*. Beveled Washer.

of wooden trestle bridges and which has given satisfactory service. It will be noted that both the diameter and height are somewhat larger than any standard washer for $\frac{3}{4}$-inch bolt. The thickness of the base plate is $\frac{1}{4}$ inch, and of the ribs $\frac{3}{16}$ inch. The spool is $\frac{5}{8}$ inch high above the base plate and is $\frac{5}{16}$ inch thick at the top. The weight is about $1\frac{1}{4}$ pounds.

Washers of the type described in the preceding paragraph were adopted in 1898 as standard in the Bridge Department of the Union Pacific Railroad, for diameters of bolts from $1\frac{5}{8}$ to $2\frac{1}{4}$ inches, the corresponding diameters of the washers ranging from 8 to 12 inches. Eight ribs were used for diameters of 10 inches and over. The base plate was also strengthened by a circular rim on the outside extending the same distance above the base plate as the thickness of the plate. For the largest size mentioned the washer is 3 inches high; the spool is 1 inch thick, and both ribs and base plate are $\frac{1}{2}$ inch thick.

Malleable-iron washers differ from cast-iron washers in being constructed with thin metal plates stiffened by radial ribs, thereby reducing the weight to one-third, and the height to about one-half of that of the corresponding cast washers. As

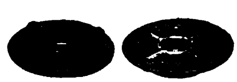


FIG. 2j.
Malleable Iron Washer.

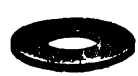


FIG. 2k.
Plate Washer.

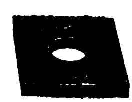


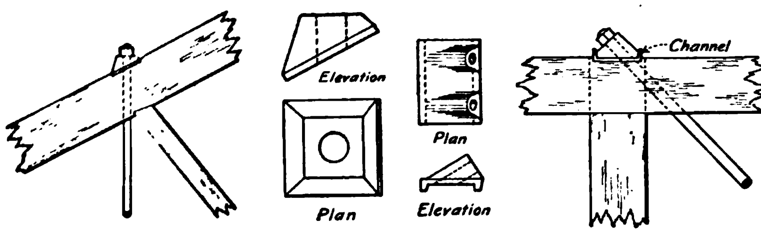
FIG. 2l.
Square Washer.

indicated in Fig 2i, the bearing area is also reduced as the bottom plates do not extend to the central cylinder. The beveled slots on top and the slots on the outer edge on other forms not shown are used to lock the nut and washer respectively with the aid of a cold chisel.

The ordinary forms of wrought-iron or steel washers are cut from comparatively thin plate, the diameter ranging from two to about three times the diameter of the bolt. The circular ones

are not intended to distribute on the wood the full tensile strength of the bolt, but the square washers are materially stronger, some sizes being proportioned for effective use on wood while others are not.

When the direction of a bolt is not perpendicular to the surface of the timber, a beveled washer is required unless only a slight notch is required to secure a square bearing. Figures *2m–p* show three forms, one a solid casting, the next a double washer



FIGS. *2m, n, o,* and *p*. Beveled Washers.

for a more acute angle between the bolt and timber surface, and the third a combination of two or more beveled washers with a channel, in order to distribute the bearing surface over the full width of the timbers. Still another form is illustrated in Fig. *2i*.

In designing a beveled washer, the bearing surfaces on the end and base must provide the proper safe resistance to the respective components of the stress in the bolt, and the resultants of these three stresses must intersect in a point if it is desired to secure a uniform distribution of the compression on the two surfaces of the timber, as illustrated on Fig. *2q*.

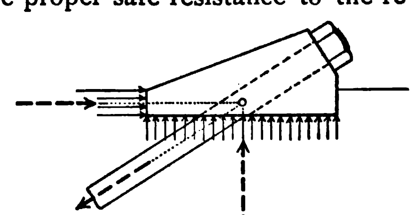


FIG. *2q*.

A few other forms of washers are used for special purposes in construction. One is the cup washer used on guard timbers of railroad tracks on bridges and elevated railroads. It is sunk

into the timber so as to prevent the bolt head from projecting above its surface. A similar form called socket washer is used in the floors of freight cars, elevators, warehouses, piers, and wharves for the same purpose. See also the reference to a double-cone washer in Art. 1.

When the bearing surface of a washer is inclined to the fibers of the wood, the relation between the allowable bearing pressure per square inch and that on the sides and ends of the fibers respectively may be obtained by the method given in Art. 62.

Prob. 2. A vertical bolt which is carrying a tensile stress of 8150 pounds passes through a timber inclined an angle of $19\frac{1}{2}$ degrees with the horizontal. The timber is $3\frac{3}{4}$ inches wide and the diameter of the upset end of the bolt is $1\frac{3}{8}$ inches. The allowable unit stresses on the sides and ends of the fibers are 2000 and 700 pounds per square inch respectively. Design the washer required, and indicate its dimensions on a pencil drawing made to scale.

ART. 3. THE STRENGTH OF WASHERS.

The standard commercial sizes of washers are supposed to resist with safety the allowable stresses in the corresponding bolts. For example, a $\frac{3}{4}$-inch bolt has an area of 0.302 square inch at the root of the thread, and for an allowable tensile stress of 15 000 pounds per square inch, it can resist a stress of 4530 pounds. The diameter of the corresponding washer in the first table in Art. 2 is $3\frac{1}{4}$ inches, and its net bearing area is $8.296 - 0.601 = 7.695$ square inches. The allowable compression on the wood must therefore not exceed $4530/7.695 = 590$ pounds. This value exceeds the average elastic limit for compression on the side of the fiber for most of the species of wood used in construction, and is about double the elastic limit for some of the softer woods so employed. If therefore a washer and a bolt of the size given are used together in structural timbers of these species, either the bolt cannot develop its proper strength, or the wood is subjected to a greater compression than should be allowed in properly designed structures. In either case there

is a theoretic lack of economy, even if no danger is directly involved. Experienced inspectors claim that, in general, evidences of weakness in timber structures, especially those exposed to the weather, are shown first in bearings on the sides of the fibers.

A manufacturer's standard square steel washer for a $\frac{3}{4}$-inch bolt is $2\frac{1}{2}$ inches square, $\frac{1}{4}$ inch thick, and has a hole $\frac{27}{32}$ inch in diameter. Its net area is 5.691 square inches, and hence the compression required to develop the working strength of the bolt is $4530/5.691 = 796$ pounds per square inch, which considerably exceeds the average elastic limit of long-leaf yellow pine and Douglas fir and is about 87 percent of that of white oak.

A series of 48 tests of cast-iron washers was made in 1909 by H. M. SPANDAU in the civil engineering laboratory of Cornell University. All of them were designed for $\frac{3}{4}$-inch bolts. Their dimensions expressed in inches are given in the following table:

DIMENSIONS OF SPECIAL CAST-IRON WASHERS.

Designation.	Diameter.	Height.	Ribs.		Base Plate Thickness.
			Number.	Thickness.	
A	3	5/8	4	3/16	1/8
B	3	5/8	6	1/8	1/8
C	3	5/8	6	1/8	1/8
D	3	5/8	4	1/8	1/8
E	3	11/16	4	1/8	3/16
G	3	9/16	6	1/8	3/16
H	3.5	5/8	6	1/8	1/8
O	2.5	9/16	6	1/8	3/16
BB	4	1	6	3/16	1/4

In all cases the vertical cylinder or spools are $\frac{5}{16}$ inch thick. The net bearing area is 4.39 square inches for the $2\frac{1}{2}$-inch washers, 6.54 for the 3-inch, 9.10 for the $3\frac{1}{2}$-inch, and 12.05 for the 4-inch washers. The weights per hundred are as follows: *A*, 53 pounds; *B*, 50; *C*, 49; *D*, 45; *E*, 55; *G*, 48; *H*, 56; *O*, 42; and *BB*, 135 pounds. Washers *C*, *E*, and *G* have the lower outer

edges rounded to a radius of $\frac{1}{8}$ inch, and washers *D* have the upper surfaces of the ribs curved to a radius of $2\frac{7}{8}$ inches to improve their appearance. All the washers except *BB* are ordinary gray castings made in the Sibley College foundry as a part of the regular output, while washers *BB* are the standard of the Bridge and Building Department of the Atchison, Topeka, and Santa Fé System (Art. 2).

Four kinds of timber were employed in the tests: Douglas fir, yellow poplar or whitewood, red oak, and white oak. The sticks

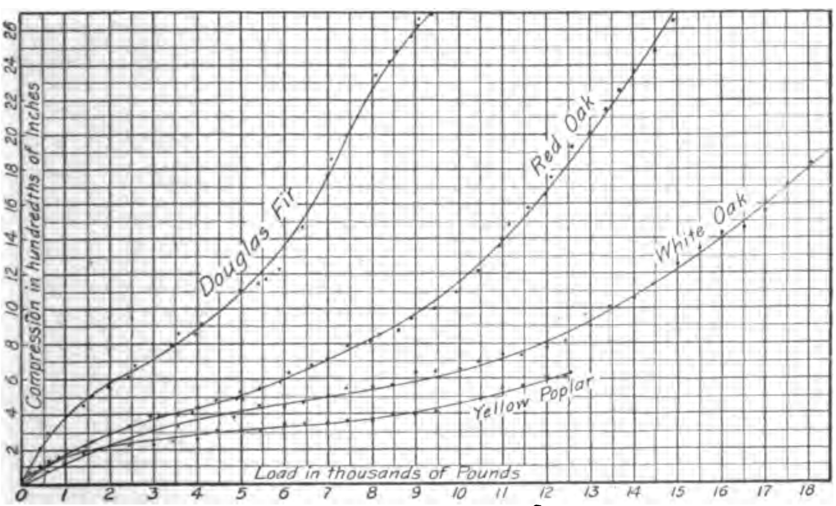


FIG. 3*a*. Typical Curves for Compression under Washers.

were from $3\frac{1}{4}$ to $5\frac{1}{4}$ inches thick. The amount of compression was measured with precision for increments of 200 pounds in loading, except for washer *BB*, where the increments were 500 pounds. Figure 3*a* gives a typical diagram indicating the relation between the load and the compression of the wooden stick under the washer. The tests were continued until the fibers of the wood broke or sheared from adjacent ones. The elastic limit was determined in every case.

Washers *A*, *D*, *H*, and *BB* were tested to destruction on red or white oak, two specimens of *H* being also broken on yellow poplar. In every case the failure occurred in the base plate between the ribs. The average ultimate resistances were 13 200, 12 250, 12 400, and 21 000 pounds respectively. Two specimens of *B* failed, while one remained unbroken at 18 000 pounds when the wood gave way. The average for *B* is 14 250 pounds. One specimen of *E* ruptured, while two remained intact when the wood yielded at 14 000 pounds. The average for *E* is 13 450 pounds. Washers *C*, *G*, and *O* failed to break in any test, although the loads averaged 15 500, 16 700 and 15 400 pounds respectively when the wood reached its ultimate strength.

Only two specimens with four ribs survived the tests, and these had base plates $\frac{3}{16}$ inch thick and rounded edges. Only one washer with a rounded edge failed. Washers *C* differ from *B* only in the rounded edge, but none of the former broke, while two of the latter broke under less load. Washers *G*, which differ from *C* by having a thicker base plate but a reduced height, are found to be stronger, thus indicating the effect of strengthening the base plate. These results show that washers *G* are the best proportioned for a diameter of 3 inches, and all of them will support more than four times the allowable stress in the bolt. The three test loads for *G*, none of the washers breaking, are 15 000, 16 900, and 18 200 pounds, the first one on red oak and the other two on white oak. The $3\frac{1}{2}$-inch washer *H* should have a $\frac{3}{16}$-inch base plate, its height remaining the same. Its edge should also be rounded, since this form permits a considerably higher load before crushing or cutting into the fibers. The tests also show that the ultimate resistance per square inch of the timber decreases as the area of the washer increases, except for yellow poplar, which is very tough wood. The same relation exists at the elastic limit with the exception of a few tests on yellow poplar. Three tests of washers *O*

ranged from 15 300 to 15 600 pounds, or an average of 3507 pounds per square inch.

Four tests were also made of a malleable iron washer. Its diameter is 3 inches, but the net bearing area is only 4.5 square inches, on account of its peculiar form, which sacrifices an annular area near the spool in order to secure a wider top. The metal is closely $\frac{3}{32}$ inch in thickness. The washers did not fracture, but the plates bent up between the ribs, of which there were, however, only three instead of four as illustrated in Fig. *2j*. The average resistance was 1300 pounds.

Since the weight of an ogee washer for a $\frac{3}{4}$-inch bolt varies from 70 to 100 pounds, these tests indicate that it is possible to design an equally strong washer with a saving of from 30 to 50 percent.

Prob. 3. Determine approximately the stress per square inch at the elastic limit of the Douglas fir and red oak in the tests indicated graphically in Fig. 3*a*.

ART. 4. NAILS AND SPIKES.

A nail consists of a slender body of metal, either tapering toward or pointed at one end, and with a head at the other end, used to drive through and into wood to fasten two pieces together, or to connect some other objects or materials in a similar manner.

Nails are classified both with respect to their method of manufacture and to their form or use. A cut nail is the common nail of rectangular cross section, which is cut from a metal strip by machinery, which also upsets and forms the head. It is called a cut nail to distinguish it from the wrought nail, which is forged either by hand or by machinery. The latter is also called a clinch nail, because its protruding point may be bent down readily without breaking, and is therefore secure against withdrawal due to shocks or vibrations. A wire nail is made directly from wire by machinery which forms both head and point.

According to their form or use, nails are known as common or ordinary, finishing, core, fence, casing, slate, light, brads, etc. Both cut and wire nails are now generally made of steel. Nails for special purposes in which corrosion, magnetic effect, and galvanic action must be avoided are made of copper, brass, and other alloys. The modern steel nails rust so much faster than the old wrought-iron nails, that it is necessary to use copper nails to secure the same life as for the material connected when exposed to the weather or to other adverse conditions.

The length of nails ranges from 1 to 6 inches, the successive sizes differing by $\frac{1}{4}$ inch up to $3\frac{1}{2}$ inches and by $\frac{1}{2}$ inch beyond

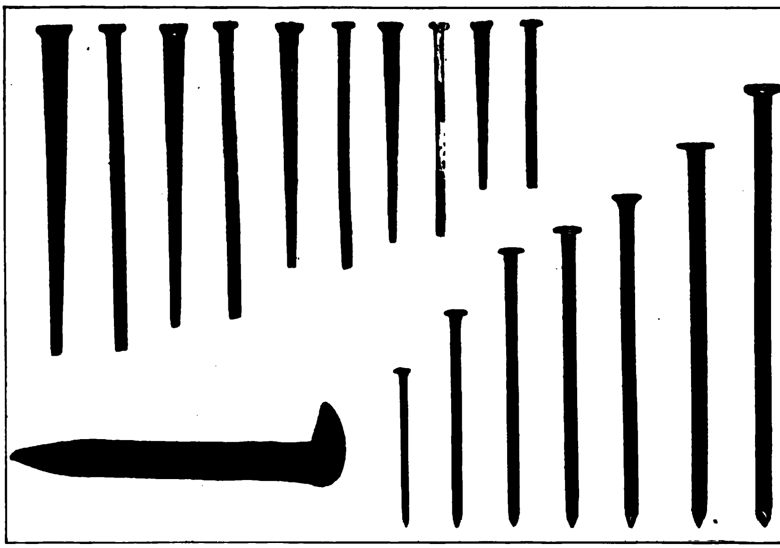


FIG. 4a. Cut Nails, Wire Nails, and Railroad Track Spike.

that limit. A nail 3 inches long is, however, not designated by its length, but as a tenpenny (written 10d) nail. This term is a corruption of '10 pounds of nail,' meaning 10 pounds to 1000 nails, which indicates the method originally adopted for measuring them. This nomenclature has lost its original significance,

for at present one manufacturer furnishes 72 common cut nails or 69 wire nails per pound, while another one furnishes 60 and 77 per pound respectively of the size known as tenpenny nails. The sizes run from twopenny to sixtypenny. The kinds and dimensions of nails are not only listed in the catalogues of manufacturers, but are given in various handbooks prepared for the use of architects, superintendents, inspectors, and builders.

The cut nail is driven so that its tapering sides bear against the bent ends of the fibers, thus acting like a wedge. The blunt point breaks and bends the fibers as it penetrates the wood. The wire nail is round, and has a pyramidal point, which separates the fibers when driven into the wood, and its resistance to withdrawal is therefore due to the bearing of a part of its cylindrical surface against the sides of the fibers. In some kinds of construction, as in freight-car building and repair work, it is found economical to use a pneumatic hammer to drive the nails after starting them by hand. In this way two men can do the work formerly requiring four men, when all of it was done by hand.

A patent cement-coated nail has been introduced in order to secure greater adhesion in the wood, and thus permit smaller sizes to be used than with the ordinary wire nail. They are made by coating wire nails with an asphaltum cement.

FIGS. 4*b* and *c*.

There are two principal classes of spikes used in timber construction; in one class the spikes have the same form as a nail, either cut or wire, and those of the other class are called boat or ship spikes. The former are manufactured in the same manner as cut or wire nails, while the latter are forged from square bars of wrought iron or steel, and have a blunt chisel or wedge point. The length of the former ranges from 3½ to 7 inches for the cut

spike and from 3 to 12 inches for the steel wire spike, while that of the latter ranges from 3 to 16 inches. The boat spikes have thicknesses of $\frac{1}{4}$, $\frac{5}{16}$, $\frac{3}{8}$, $\frac{7}{16}$, $\frac{1}{2}$, and $\frac{5}{8}$ inches, while the length varies for each thickness, being from 3 to 7 inches for the smallest, and from 8 to 16 inches for the largest thickness. The point is wedge-shaped, the length of the taper being about twice the thickness of the spike (Fig. 4*b*). The edges should not be ragged, as indicated in the illustration, since it reduces the holding power (Arts. 5 and 7). The railroad track spike used to fasten the steel rail to a timber cross-tie resembles the boat spike in general form, except in its eccentric head, its largest size being $\frac{5}{8}$-inch thick and 6 inches long (Figs. 4*a* and 4*c*).

Prob. 4. Get a 20d and a 60d cut nail and measure the thickness and width at the point, at the middle, and directly under the head; also the diameter of wire nails of the same size.

ART. 5. HOLDING POWER OF NAILS.

A carefully arranged set of experiments on the relative holding power of nails under different conditions was made by F. W. CLAY in the laboratory of the College of Civil Engineering, Cornell University, the results of which are published in Engineering News, vol. 31, page 22, Jan. 11, 1894. The most important facts and principles obtained from these tests are given in the following paragraphs.

By driving a cut nail in a strip of wood between two holes it was found that the resistance to withdrawal is less than 8 percent smaller than when driven into the same stick in the usual manner. As this difference is less than that frequently obtained for two nails driven under the ordinary conditions, it may be said that the holding power of a cut nail depends only on the friction on the wedge or taper sides. As the nail penetrates the wood the blunt point breaks and bends the fibers and then compresses them as it is driven deeper. On withdrawal,

these fibers tend to straighten out and increase their pressure on the rough surface of the taper sides. The other sides of the nail are parallel and smooth and develop but little or no resistance.

Tests made on six different forms of point indicated their effect upon the holding power of the cut nail. The highest result was given by a short pyramidal point, the resistance being 32 percent greater than for the customary blunt end. This increase in holding power appears to be due both to a reduction in the number of fibers that are injured by breaking, and to separating the remaining fibers so as to develop pressure and friction on the parallel sides of the nail. If, however, this form of point is materially lengthened, it will merely separate and compress the fibers, thus eliminating the wedge action of the taper sides, and reduce the holding power below that for the blunt point used in practice.

If the ordinary cut nail is driven with its taper sides parallel to the fibers of the wood, the resistance to withdrawal is larger than when driven in the usual manner. This difference is much smaller in the hard woods than in the soft ones. The tendency to split the timber is materially increased at the same time.

The wire nail with its ordinary pyramidal point forces apart the fibers and develops pressure on the sides of the fibers, the surface of contact being less than the full cylindrical surface of the nail. If the wire nail had a blunt point, its holding power would be less than half as great. The blued surface which is slightly granular and rough increases the holding power and soon rusts fast to the wood, but the process employed injures the quality of the nail, so that many are spoiled in driving.

Comparative tests showed that the ratio of the initial holding power to the weight is larger in every case for the cut nail than for the wire nail. The effect of sharp barbs is to reduce the resistance of both cut and wire nails.

When the cut nail is withdrawn, its resistance decreases very rapidly. When withdrawn about 5 percent of its penetration, or fairly started, only about 60 percent of its holding power remains. Its holding power decreases more rapidly than that of the wire nail, and under vibration the cut nail jars loose sooner than the wire nail. The proper length for a nail to resist direct withdrawal is about 3 times the thickness of the thinner piece nailed.

The absolute value of the holding power of a nail depends not only upon the form, length, and kind of nail, but also upon the kind of wood, its state of seasoning, thickness of the annual rings, the proportion of hard summer wood in the rings, the relation of the nail to the rings and to the fibers, the mode of driving and of withdrawing the nail. On account of the large number of these elements and the considerable variation of some of them, any published tables of absolute holding power must be used with caution, especially when the conditions of the tests are not fully described.

As the result of a challenge by the wire-nail manufacturers to the cut-nail manufacturers, a series of experiments was made at the Watertown Arsenal in December, 1892, and January, 1893, including tests of 58 groups, each group including 10 cut nails and 10 wire nails of the same size. A summary of the results was published in the Railroad Gazette, vol. 25, page 356, May 12, 1893. At first forty groups were tested, including various sizes of common, light common, finishing, box, and floor nails, all driven in spruce wood, and it was found that the superiority of the cut nails over the wire nails in holding power for the classes mentioned was 47.5, 47.4, 72.2, 50.9, and 80.0 percent respectively, or an average of 60.5 percent for all of these tests. After that, 18 extra groups of box nails were tested in pine with different relations to the grain of the wood, and in this case the superiority ranged from 64.4 to 135.2 percent, or an average of

99.9 percent. The least value of 64.4 percent applies to nails driven parallel to the fibers.

While these experiments apparently proved that in every case the cut nail was superior to the wire nail in holding power, it must be remembered that the holding power considered is the resistance at starting to pull the nail, and not that still remaining after the nail has been partially withdrawn, the basis of comparison being the area of the surface of the nail in contact with the wood. As subsequently shown in this article, the holding power decreases much more rapidly in the cut nail on account of its wedge form, and a comparison of that remaining indicates the superiority of the wire nail for most purposes. Experience has shown that boxes nailed with wire nails stand handling in transportation better than those made with cut nails, especially for the heavier commodities.

RESISTANCE AND WORK IN DRIVING AND PULLING NAILS IN YELLOW PINE.

Size and Kind of Nail.	Number of Nails per Pound.	Depth of Penetration in Inches.	Maximum Load in Pounds for Driving.	Maximum Load in Pounds at Start in Pulling.	Work per Nail in Inch Pounds.		Ratio of Work in Pulling to Driving.
					To Drive.	To Pull.	
20d cut	23	3 5	820	921	1523	478	0 32
20d wire	34	3.5	376	318	865	473	0.55
10d cut	70	3.0	342	357	585	201	0.36
10d wire	105	3.0	232	214	436	220	0.51
8d cut	88	2.25	312	328	419	140	0.33
8d wire	132	2.25	199	167	279	105	0.38
6d cut	168	1.75	221	156	274	65	0.24
6d wire	252	1.75	135	88	165	63	0.38

The work performed as well as the force required in both driving and pulling cut and wire nails were investigated by W. E. BARNES and R. O. STILLWELL in the Sibley College laboratory of Cornell University, the detailed results of which may be

found in a paper by R. C. CARPENTER in Transactions American Society of Mechanical Engineers, 1895, vol. 16, page 1002.

Nails of different sizes were forced into a piece of southern pine, which was as nearly homogeneous as it was possible to obtain, by one of the heads of a testing machine, and the force required at the end of each one-quarter-inch of penetration was noted. In pulling them the force required was again noted at

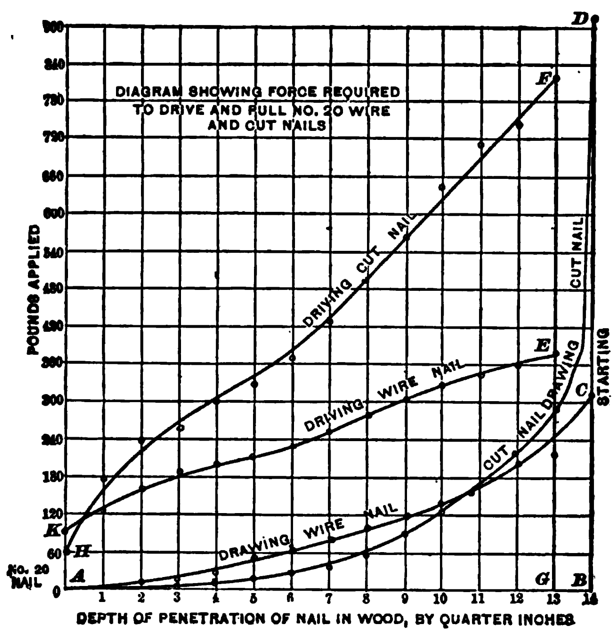


FIG. 5a. Resistance to Penetration and Withdrawal of 20d Nails.

the beginning of each quarter-inch withdrawal. Ten nails of each size and kind were tested and the averages taken in plotting the diagrams.

A general summary of the results is given in the preceding table. From the table and diagrams here reproduced it will be noted that they confirm the previous statements based on other experiments with reference to the relative maximum holding

power of cut and wire nails, and the rate of reduction in the resistance as the nails are withdrawn. They also show that the total work in inch-pounds per nail in driving cut nails is much more than for wire nails. The work required in pulling nails is however nearly equal for the two kinds of nails, being sometimes larger for one and then for the other. If the work per nail be multiplied by the number of nails per pound, it will be observed

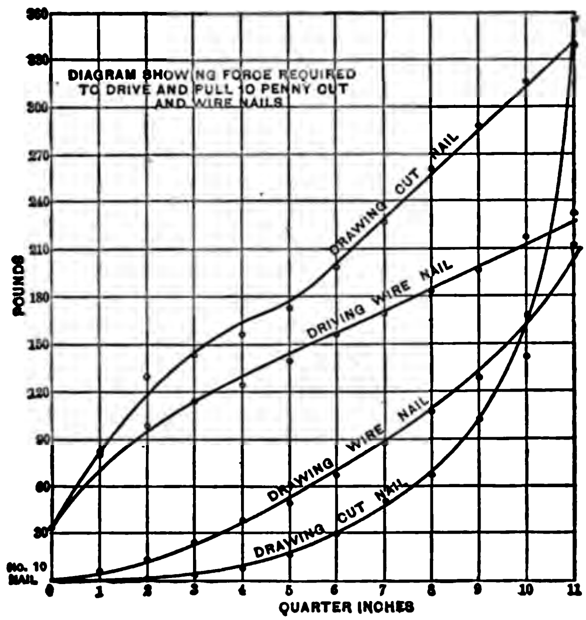


FIG. 5*b*. Resistance of 10d Nails. [On upper curve for drawing read driving.]

that the work required per pound of nails is less for wire than for cut nails in driving, but considerably greater in pulling. The ratio of the work of pulling to that of driving is also much higher for the wire than for the cut nail.

As previously stated a nail in tension should have a length equal to about three times the thickness of the piece held in

place by the nail. The holding power then depends upon two-thirds of the length of the nail from its point. The diagrams show that in this case the holding power is somewhat larger for the wire nail than for the cut nail, while the relative difference in the total work required to withdraw that part of the nail is still greater and also in favor of the wire nail. The student

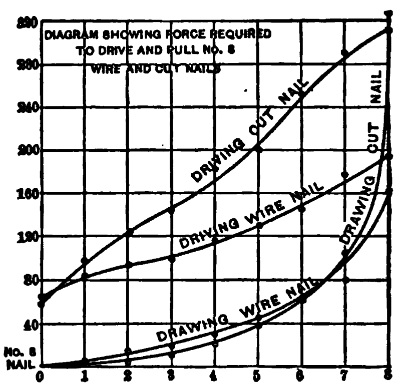


FIG. 5*c*. Resistance of 8d Nails.

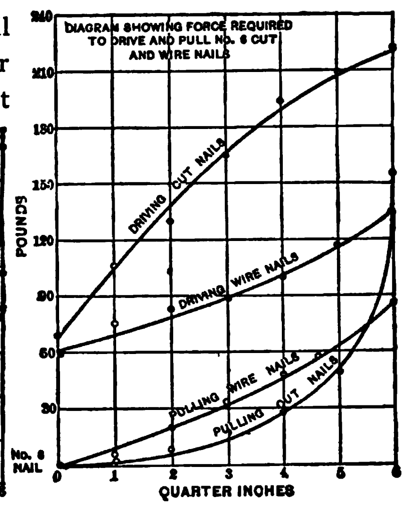


FIG. 5*d*. Resistance of 6d Nails.

is referred to the discussion of CARPENTER'S article, in which important facts and experience are given on the use of different kinds of nails for various purposes to which they are adapted.

The results of a series of experiments on the holding power of nails in Douglas fir and redwood, made by FRANK SOULÉ in 1896, are published in the Journal of the Association of Engineering Societies, vol. 20, page 115, February, 1896. A few of the more important conclusions are as follows: The cut nail is superior to the wire nail when driven into Douglas fir, but in redwood the wire nail is superior to the cut nail. The redwood is softer than the Douglas fir, but the former had 39 rings, while the latter had 14 rings per inch. This fact bears out the theory as to the manner in which the wire nail holds. The holding

power of nails increases with time in redwood, but decreases in Douglas fir. In both kinds of wood the holding power is about three times as great when the nails are driven perpendicular to the fiber as when driven parallel to the fiber. In Douglas fir, cement-coated nails have about 30 percent greater holding power than plain wire nails, but in redwood the difference is small. The following average ultimate resistances, expressed in pounds per square inch, were obtained: Cut nails in Douglas fir, 481; cut nails in redwood, 282; wire nails in Douglas fir, 296; wire nails in redwood (3 tests only), 367.

A series of tests on the adhesive resistance of nails in five kinds of timber was made at the Watertown Arsenal, the record of which is published in Tests of Metals, etc., 1884, page 448. The average holding power for cut and wire nails when driven in the usual manner is given in the accompanying table:

ULTIMATE HOLDING POWER OF CUT AND WIRE NAILS.

(Expressed in pounds per square inch.)

Kind of Wood.	Direction of Nail to Grain.	Cut Nails. 8, 9, 10, 20d.	Cut Nails. 50, 60d.	Wire Nails. 10d.
White pine	Perpendicular	451	350	167
	Parallel	223	192	126
Yellow pine	Perpendicular	685	636	318
	Parallel	578	483	252
White oak	Perpendicular	1277	1072	940
	Parallel	1134	977	802
Chestnut	Perpendicular	——	683	——

and is expressed in pounds per square inch of the surface of contact between nail and wood. The depth of penetration used did not exceed two-thirds of the length of the nail, so as to conform to average conditions in practice. Four tests were made for each size of nail, except the 50d and 60d, for which the number was doubled.

It was found that when the cut nail is driven with the taper acting across the grain, thus tending to split the timber, the

average resistance for all the sizes mentioned above is increased 33 percent in white pine, 21 percent in yellow pine, 14 percent in white oak and 5 percent in chestnut, or an average of 18 percent. The effect of barbing the cut nail is to diminish its average holding power in these four kinds of wood about 15 percent, the reduction being the least in the soft wood, like white pine.

When sticks of wood, in which 60d nails had been driven, were exposed to tide-water and air alternately for 13 days, until they were water-soaked as deep as the nails penetrated, the adhesive resistance of the nails was found to be reduced 43 percent in white pine and 23 percent in yellow pine and white oak. Nails of the same size were then driven into the wet sticks and stored in a cool, dry building for 6.8 years. Upon being tested the resistance of the nails was still found to be 43, 53, and 9 percent below the original values for the three kinds of wood respectively (see Tests of Metals, 1891, page 744).

The cement-coated nails described in the previous article have a much larger adhesive resistance than smooth wire nails. The following table gives the results of comparative tests made in 1902 at the Watertown Arsenal. The nails were driven across the grain into pine timber, the heads being allowed to project $\frac{1}{4}$ to $\frac{1}{2}$ inch. Each value is the average of three tests.

COMPARATIVE HOLDING POWER OF SMOOTH AND COATED WIRE NAILS.

Size	6d	8d	9d	10d	
Depth driven	$1\frac{5}{8}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	inches
Cement-coated	226	316	327	418	pounds
Common smooth	106	189	182	167	pounds
Ratio	2.13	1.67	1.80	2.50	

The holding power of wire nails in different kinds of wood may be assumed to be approximately proportional to their resistance to compression on the side of the fiber.

Prob. 5*a*. From the data in the preceding table of holding power of cut and wire nails, based on the Watertown tests, compute the percentage of the resistance of each kind of nail when driven parallel to the fiber to that when driven perpendicular to the fiber.

Prob. 5*b*. Compare the numerical results obtained by CLAY and SOULÉ for the initial holding power of both cut and wire nails, with different forms of points.

Prob. 5*c*. Referring to CARPENTER's paper, plot the curves for each size of nail, showing both the maximum and minimum, instead of the average values of the holding power.

Prob. 5*d*. Plot on the same diagram the resistance to pulling an ordinary 10d cut nail and one with its point sharpened, as given by CARPENTER, to show the effect of sharpening the point. Find also the work done in each case.

ART. 6. LATERAL RESISTANCE OF NAILS.

The most extensive series of experiments on the strength of nails to resist longitudinal shear in a lap joint (Art. 27), consisting of a cleat nailed to a block, was made in 1897 by F. B. WALKER and C. H. CROSS in the engineering laboratory of the University of Minnesota. The results may be found in the Journal of the Association of Engineering Societies, vol. 19, page 260, December, 1897. About 300 tests were made on various sizes of wire nails, including a few on spikes 7 and 8 inches long. The wood used was Norway and white pine, except for a few tests in which oak was employed.

The illustrations in that article show that for ultimate tests the fibers on the adjacent planed surfaces of the timbers crush against the nail, which bends into a reverse curve. In no case does this crushing extend to the middle of the thinner timber. In the oak the bends are very short and quite symmetrical. In the 20-penny nails they vary in length from 1⅜ to 1½ inches for the same extension or slip of ⅞ inch in the joint. In such a joint, therefore, the nails are not subject to a true shear but rather to flexure. There seems to be a fairly constant relation between the diameter of the nail and the length of the bend.

Upon removing the load which causes a longitudinal extension of $\frac{1}{16}$ inch the joint recovers completely, but if the extension exceeds that amount, the recovery is only partial, ranging from $\frac{1}{32}$ to $\frac{1}{8}$ inch. This indicates approximately the yield point of the joint. The elastic limit is practically one-half of the ultimate strength of the joint. The extension under maximum load is apparently independent of the size of the nail.

If the nail penetrates the second timber less than half, but not less than one-third, of its length the ultimate strength of the joint is reduced but the elastic limit remains the same. In experiments where the number of nails varies from 2 to 10 it is found that the strength of the joint is directly proportional to the number of nails, thus indicating that they act together in an effective manner.

The timbers used were white pine, Norway pine, and oak, the mean compressive strength for short blocks being 4840, 5820, and 6600 pounds per square inch respectively. Although the compressive strength of Norway pine is 20 percent greater than white pine, there is practically no difference between the shearing strengths of the corresponding nailed joints. On the other hand, the compressive strength of the oak is only 36 percent greater than that of white pine, but the corresponding difference in the nailed joints is from 110 to 130 percent greater. This large difference appears to be caused by the difference in wood structure of oak and pine, and is fairly measured by their difference in resistance to compression on the side of the fiber.

To prevent splitting $2\frac{7}{8}$-inch Norway pine with 50- and 60-penny nails they have to be placed $1\frac{1}{2}$ inches from the end and 1 inch from the edge; while with 6-penny nails and a 1-inch board, the corresponding distances are $\frac{3}{4}$ and $\frac{1}{2}$ inch respectively. Where the board or plank is not split the full strength of the nail is developed independently of its position, the same as if located in the center of the piece.

In the discussion of these tests, A. W. MUENSTER, at whose suggestion they were made, proposed to express the shearing strength per nail of the joint at its yield point by the provisional formula Cd^2, in which d is the diameter of the wire nail and C a coefficient depending on the kind of wood employed. For white and Norway pine its value may be about 5500, varying probably from 4000 for green sap timber to 6500 for thoroughly seasoned wood. The coefficient for oak as deduced from the few experiments made is about 13 500, but the oak used was considerably better than the average, being so thoroughly seasoned and so hard that it was impossible to drive into it any nail above 20-penny size without previously drilling a hole. It was also proposed to use an allowable stress of from 80 to 60 percent of the elastic limit, depending upon the more or less temporary character of the structure. The above formula may be applied to wire nails from 2 to 7 inches in length.

The following table gives a comparison between the experimental value of the elastic limit averaged from all the tests and the corresponding values of the formula $5500\,d^2$. Numerically these values are low in comparison with the other tests referred to in this article. The article from which these results are obtained contains ten excellent half-tone illustrations showing the effect of the penetration of nails and spikes upon the fibers of the wood, and their curvature due to the sliding of the cleat on the block.

STRENGTH OF WIRE NAILS AT ELASTIC LIMIT OF JOINT FOR WHITE AND NORWAY PINE.

Size of nail	6d	8d	10d	16d	20d
Experimental value	55	88	112	112	218 pounds
Value by formula	67	90	127	120	212 pounds
Size of nail	30d	40d	50d	60d	80d
Experimental value	226	275	342	362	500 pounds
Value by formula	231	280	324	373	506 pounds

The time test reveals a certain extension of the joint under the load, called the yield point for a single application after a sufficient lapse of time, but this movement will not exceed probably $\frac{1}{20}$ to $\frac{1}{40}$ inch and can hardly be considered of any moment in ordinary wooden construction.

A systematically designed series of experiments was made in 1898–99 by H. D. DARROW and D. W. BUCHANAN in the engineering laboratory of Purdue University, the results of which were presented to the Indiana Engineering Society by W. K. HATT and published in the Proceedings for 1900. Approximately 250 joints were tested, observing the slip and corresponding load at about 12 different points for each joint. Well-seasoned yellow pine and oak wood were employed and the thickness of the cleats was varied to cover ordinary practice. Unless otherwise stated the grain of the wood in both cleat and block was placed parallel to the direction of the applied load. Both wire and cut nails were tested, the sizes and kinds included being 2d to 60d common, 4d to 12d finishing, 8d and 10d fence, and 3d fine.

It was found that a slip of $\frac{1}{16}$ inch corresponds to the yield point of the nailed joint. Up to this point the elasticity of the joint enabled it to recover fully when the load was removed. The slip under maximum load varied considerably, and for 10d wire nails was 1¼ inches. About 10 percent of the larger sizes of cut nails broke, but in all very few nails failed in this manner.

For tests with 8d and 10d wire nails and from 1 to 6 nails in the joints, the strength per nail is practically constant both at the yield point and at its ultimate value.

To determine the effect of the length of nail, 14 yellow pine joints were tested, having 1⅛-inch cleats nailed to the blocks with 16d common wire nails cut to different lengths, varying by ¼ inch, from 2 to 3½ inches. At the yield point the resistance was 352, 392, 355, 410, 412, 415, and 367 pounds, or an average of

386 pounds. Each value is the mean of two tests. As the extreme range of these values is only one-half as large as the greatest difference between two tests for the same length, the strength at the yield point may be regarded as practically the same. The ultimate strength is nearly proportional to the length of the nail, decreasing a little in proportion for the longer nails, the average load per linear inch of nail being 238, 229, 213, 222, 217, 208, and 192 pounds.

The effect of changing the diameter of nail but not the length was investigated with cleats $\frac{7}{8}$ inch thick and wire nails 2 inches long. The results are given in the accompanying table, which indicates that the larger sizes are relatively at a disadvantage:

Nominal size	6d	10d	16d	20d	
Diameter	0.112	0.15	0.16	0.187	inches
At yield point	224	352	417	402	pounds
Ultimate strength	327	462	507	660	pounds

The best angle at which to drive the nail is at right angles to the surface of contact between the timbers. On inclining the nails in either direction the resistance is reduced. The effect of barbing the nails and of soaking the joint is to decrease the strength of the joint.

By changing the direction of the grain so as to make it perpendicular to the direction of the applied load in either cleat or block, or in both of them, the resistance remained practically the same. When, however, the fibers of the block were so placed that the nails were driven parallel to the fibers, the resistance decreased at least one-third.

The use of various combinations of yellow pine and oak in cleat and block made no decided difference in strength; when a white pine cleat was used with a yellow pine or oak block, there was a noticeable reduction in strength at the yield point, but not in the ultimate strength; but when a white pine block was used the reduction was about 50 percent at both yield point

and ultimate resistance. The white pine had an ultimate compressive strength parallel to the grain of 6300 pounds per square inch.

The strength of various sizes and kinds of steel nail is given in the accompanying table. Two sticks of yellow pine were used in the tests, having an ultimate compressive strength parallel to the grain of 7000 and 10 200 pounds per square inch respectively, the former being used in testing all nails not larger than 8d, while the latter was used for the other sizes. The compressive strength of the oak was 5300 pounds per square inch.

LATERAL RESISTANCE OF NAILS.

(Expressed in pounds.)

Kind of Nail.	Size.	Yellow Pine.		Oak.		Kind of Nail.	Size.	Yellow Pine.		Oak.	
		Wire.	Cut.	Wire.	Cut.			Wire.	Cut.	Wire.	Cut
Common	2d		130		160	Common	60d	2000	1860	1770	
Common	3d		180	184	289	Finish	4d	106	163	186	262
Common	4d	198	213	211	344	Finish	6d	216	209	299	273
Common	6d	240	317	314	429	Finish	8d	264	282	359	405
Common	8d	361	427	454	573	Finish	10d	537	451	583	
Common	10d	724	932	762	822	Finish	12d	498		637	
Common	16d	855	1079	891	1006	Fence	8d	690	686	709	780
Common	20d	930	1112	1350	1631	Fence	10d	855	912	1030	1092
Common	40d	1450	1360	1745	1874	Fine	3d	121		164	

The strength is fairly proportional to the penny designation of the sizes up to 20-penny nails. The load per nail in the joint may be expressed in pounds with sufficient precision for the purpose of design by $P = Cd$, in which d denotes the size and C is a constant given in the following table:

Kind of wood	Oak		Yellow Pine	
Kind of nail	Cut	Wire	Cut	Wire
Common	82	70	50	48
Finish	49	52	42	32
Fence	100		90	84

The joint will slip at about 60 percent of the above loads. The safe load to adopt in designing is rather determined with reference to the yield point than the ultimate resistance.

The strength was also found to vary closely in proportion to the surface of the nail. At the yield point the load per nail in pounds may be expressed as follows, A being the area of the surface in square inches: for cut nails in oak, $555A$; for cut nails in yellow pine, $430A$; for wire nails in oak, $315A$; and for wire nails in yellow pine, $255A$. The corresponding values of the ultimate strength are $785A$, $750A$, $540A$, and $420A$. These formulas fit the tests fairly well up to 40d nails, except for cut nails in pine, for which the limit is 20d.

In comparing the resistance per pound of nails it is found that the smaller nails hold more than the larger ones. The wire nails hold more than cut nails in 12 out of 14 cases, when driven into yellow pine, and in 8 out of 13 cases when driven into oak. The 10d common and finishing nails seem to be better proportioned than the 6d or the 8d.

In some experiments on nailed lap joints made by CLAY (see reference in preceding article) it was found that the cut nail has a greater resistance than the wire nail in soft wood, while in hard wood the wire nail is superior, the difference being, however, comparatively small. He also found that the strength per unit of surface in the wood was approximately constant.

In SOULÉ's experiments (see preceding article) are included some tests on nailed joints, using both Douglas fir and redwood. It was found that the strength of the joint is practically the same for both kinds of wood, but that the cut nails develop 1.4 times the strength of wire nails of the same nominal size.

It has been previously stated that the ultimate strength of the nail under a shearing load is proportional to the square of its diameter for a given ratio of its length to the thickness of the cleat; that it is also proportional to the area of contact

with the wood; and that it increases in direct proportion to the length of the nail. These facts seem to indicate that the ultimate resistance is a combination of flexure and tension. Below the yield point, however, it is probably pure flexure, but with an unequally distributed bearing on its surface, being the greatest next to the surface of contact between the timbers.

Some experiments on the dynamic strength of iron cut nails were made in 1890 by W. C. RIDDICK in the civil engineering laboratory of Lehigh University. With the nails to be tested a board was nailed to a heavy joist placed on a firm foundation so that the top of the board projected 3 or 4 inches above the joist. A wooden ram weighing 44 pounds, fitting loosely in wooden guides, was used to find the dynamic energy required to break the nails in the joint.

The kind of wood into which the nails were driven seemed to make no difference in the dynamic strength developed. In order to compare the dynamic with the static strength the latter was observed for nails driven into both hemlock and oak. The thickness of the board was usually about one-third of the length of the nail. In the case of the smaller nails the heads were often pulled through the board under the static test. The results are briefly summarized in the accompanying table. The record of the experiments was originally published in the Journal of the Engineering Society of Lehigh University, vol. 5, page 117, June, 1890.

ULTIMATE DYNAMIC AND STATIC STRENGTH OF CUT NAILS IN SHEAR.

Size of nail	4d	6d	8d	10d	12d	20d
Number of tests	9	8	16	11	5	6
Dynamic strength	13	21	35	38	67	90 foot-pounds
Number of tests	10	6	6	13	7	2
Static strength in hemlock	185	276	302	416	366	679 pounds
Number of tests	7	3	7	9	3	2
Static strength in oak	287	419	390	439	524	685 pounds

The practical significance of these results may be illustrated by a single example. Neglecting losses due to impact, it takes about 36 foot-pounds to break an 8-penny nail. If a man weighing 150 pounds dropped three feet to the floor on a scaffold, he would break $\frac{450}{36}$ or nearly 13 nails, while the same number of nails could bear an ultimate static load equal to the weight of 26 men. The number of nails must be divided by a suitable factor in order to resist with safety the effect of the fall. This example may indicate the cause of accidents to workmen on scaffolds, to the construction of which too little thought is often given.

Prob. 6*a*. Discuss the tests of nails in joints under a shearing load, and state the facts and arguments on which to base their comparative strength for different species of wood. On what physical property of the wood does the strength of nailed joints depend ? Consider each property in turn, giving arguments for and against.

Prob. 6*b*. Show the relations between the lengths of wire nails and their lateral resistance in oak wood with the aid of a diagram.

ART. 7. HOLDING POWER OF SPIKES.

A series of 27 tests were made on the holding power of common spikes at the Watertown Arsenal, the results being recorded in Tests of Metals, etc., 1884, page 448. All the spikes are ½ inch in section and 8 inches long, including a wedge point 1 inch long, and were driven from 4 to 6 inches into the wood. The average resistance is given in pounds per square inch of the surface of contact between wood and spike exclusive of its wedge point.

ULTIMATE HOLDING POWER OF COMMON SPIKES.

SIZE OF SPIKE.	KIND OF WOOD.	NO. OF TESTS.	RESISTANCE. lb. per sq. in.	RESISTANCE. lb. per lin. in.
1/2 inch	White pine	4	224	448
1/2 inch	Yellow pine	8	530	1060
1/2 inch	White oak	4	764	1528
1/2 inch	Chestnut	3	720	1440

Two tests made by driving the spike parallel to the grain in white pine gave a resistance of only 70 pounds per square inch. The remaining tests were made to determine the effect of water soaking the timber to the depth of penetration of the spikes. The resistance was reduced 40 percent in white oak, 31 percent in yellow pine, while in white pine no reduction was observed.

In the Report of the Chief of Engineers, U.S.A., 1884, pages 2064 and 2065, may be found the detailed record of tests made by NOBLE and GILBERT to determine the adhesive resistance of boat or ship spikes. The sizes tested are $\frac{3}{8}$, $\frac{7}{16}$, and $\frac{1}{2}$ inch, the spikes being driven 3, 4, 5, and 7 inches into white pine wood. The results are summarized in the following table:

ULTIMATE HOLDING POWER OF BOAT SPIKES PER LINEAR INCH.

SIZE OF SPIKE.	KIND OF WOOD.	NO. OF TESTS.	ADHESIVE RESISTANCE.	DIRECTION OF EDGE OF POINT.
3/8 inch	White pine	18	370 pounds	Across the grain
3/8 inch	White pine	18	450 pounds	Parallel to grain
7/16 inch	White pine	10	344 pounds	Across the grain
7/16 inch	White pine	10	436 pounds	Parallel to grain
1/2 inch	White pine	2	429 pounds	Across the grain
1/2 inch	White pine	2	521 pounds	Parallel to grain

The average resistance with the edge of the point parallel to the grain is about 23 percent greater than with the edge across the grain. When it is remembered that the allowable working stress on the side of the fibers is less than one-sixth of that on the end of the fibers, the relation given in the preceding sentence shows to what extent the fibers must be injured when the spikes are driven with the edge of the point across the fibers and no hole is previously bored.

To determine the resistance of a boat spike when pulled in the direction in which it is driven, 12 tests were made of $\frac{3}{8}$-inch spikes driven into a plank of white pine 3 inches thick until the head was flat with the surface. It required an average force of 1521 pounds to pull a spike with its head through the plank.

The average resistance for 12 tests of $\frac{1}{2}$-inch spikes was similarly found to be 1708 pounds.

Assuming that the resistance of the body of the spike in the direction of driving is 60 percent of the resistance to withdrawal, as determined for drift bolts (see Art. 10), the resistance due to the head of a $\frac{3}{8}$-inch spike is approximately $1521 - (3 \times 0.60 \times 370) = 855$ pounds, while that for a $\frac{1}{2}$-inch spike is $1708 - (3 \times 0.6 \times 429) = 946$ pounds.

The holding power of the common railroad spike in wooden cross-ties is given by W. KENDRICK HATT in Circular 46 of the U. S. Forest Service, issued Dec. 26, 1906. The size of the spikes used in the tests is $\frac{9}{16}$ inch square and $5\frac{1}{2}$ inches long under the head, being driven 5 inches into the wood in all cases. The average adhesive resistance was found to be as follows: for 5 tests in partially seasoned white oak, 6950 pounds; 5 tests in seasoned oak (probably red oak), 4342 pounds; 28 tests in seasoned loblolly pine, 3670 pounds; 12 tests in green hardy catalpa, 3224 pounds; 11 tests in green common catalpa, 2887 pounds; and 4 tests in seasoned chestnut, 2980 pounds. A knotty cross-tie has about 25 percent less holding power for common spikes than one with clear wood. The circular also gives the results of tests with loblolly pine cross-ties which were steamed, or soaked, or steamed and treated with either creosote or zinc chlorid. The treating process reduced the holding power of the spikes.

The same circular also gives the holding power of the screw spike, showing that its resistance to withdrawal is from two to three times that of the common spike. A discussion of the use and advantages of screw spikes with numerous illustrations is given in Bulletin No. 50 of the Bureau of Forestry, issued in 1904, and entitled Cross-tie Forms and Rail Fastenings, with Special Reference to Treated Timbers, by HERMANN VON SCHRENK. See also an article on The Question of Screw

Fastenings to secure Rails to Ties, by W. C. CUSHING, in Proceedings of the American Railway Engineering and Maintenance of Way Association, 1909, vol. 10, page 1456.

An elaborate series of experiments on the holding power of railroad spikes was made at the University of Illinois Engineering Experiment Station, the results being given in Bulletin No. 6, by R. I. WEBBER, published in 1906. Almost the entire bulletin was reprinted in Railroad Age Gazette, vol. 43, page 143, Aug. 9, 1907. About 400 tests were made on the common railroad spike and nearly 100 on the screw spike; 44 full-size cross-ties were used, 31 of which were treated by one of three different preservative processes. In all cases the common spikes were driven to a depth of 5 inches into the cross-ties by an experienced track foreman. The entire series of tests was systematically planned, and the results thoroughly analyzed and discussed. A summary of the principal results and conclusions is given in the following paragraphs. A considerable number of additional tests was made on the lateral resistance of railroad spikes, which are referred to in Art. 8.

In most cases the resistance was measured for withdrawals of $\frac{1}{8}$, $\frac{1}{4}$, $\frac{1}{2}$, and $\frac{3}{4}$ inch, respectively, as well as for the distance which gives the maximum resistance. Three tests were made for each given set of conditions, involving size of spike, form of point, kind of wood, method of treatment, etc. A single test selected at random gives the following conditions and results: the cross-tie is a seasoned and untreated white oak wood; the spike is $\frac{5}{8}$ inch square in section, nominally 6 inches long, and has a chisel point; the distances of withdrawal and the corresponding resistances are $\frac{1}{8}$ inch, 3190 pounds; $\frac{1}{4}$ inch, 5260 pounds; $\frac{1}{2}$ inch, 3810 pounds; $\frac{3}{4}$ inch, 3500 pounds; and the maximum resistance is 6410 pounds, the corresponding withdrawal being $\frac{3}{16}$ inch.

While in a few individual tests the resistance is greater for a

withdrawal of $\frac{1}{8}$ inch than for $\frac{1}{4}$ inch, the numerous averages of 3 tests each give a smaller resistance for $\frac{1}{8}$ inch with one exception, which occurs in an untreated loblolly pine cross-tie. In practice, cross-ties of this species of wood should always be treated.

AVERAGE HOLDING POWER OF RAILROAD SPIKES IN CROSS-TIES.

KIND OF WOOD, TREATED OR UNTREATED.	SPIKES TESTED.	RESISTANCE FOR $\frac{1}{8}$-INCH WITHDRAWAL.	RESISTANCE FOR $\frac{1}{4}$-INCH WITHDRAWAL.	MAXIMUM RESISTANCE.	CORRESPONDING WITHDRAWAL.	RELATIVE RESISTANCE.		
						$\frac{1}{8}$ Inch	$\frac{1}{4}$ Inch	Max.
Untreated:	No.	Pounds.	Pounds.	Pounds.	Inch.	Percent.	Percent	Percent
White oak	30	3510	3950	7870	5/16	100	100	100
Elm	33	2310	5390	7290	3/8	66	136	93
Beech	9	2240	3790	8180	3/8	64	96	104
Chestnut	12	2990	4070	5190	3/16	86	103	66
Loblolly pine	6	2920	3190	3630	3/16	83	81	46
Treated:								
Water oak	48	2870	5730	6780	5/16	82	145	86
Black oak	39	2910	5890	7230	5/16	83	149	92
Red oak	60	2950	5350	7730	5/16	84	135	98
Burr oak	9	2670	5450	9210	3/8	76	145	117
Ash	6	3570	5200	7730	5/16	101	131	98
Elm	15	2590	5940	7500	5/16	74	150	96
Beech	9	2950	6190	8900	3/8	84	157	113
Poplar	12	2830	5290	5670	5/16	81	134	72
Loblolly pine	12	2920	3780	4310	1/4	83	96	55
Sweet gum	15	3230	5320	5300	3/16	92	103	67

The general averages for the different timbers, both treated and untreated, are given in the accompanying table. It is observed that the general effect of treating the cross-ties is to increase the holding power of the spikes. The maximum resistances of 15 spikes in 3 untreated cross-ties of red oak were also obtained, the average being 6460 pounds, indicating that the effect of treatment for that wood is to increase the maximum resistance 20 percent.

In pulling the spikes from the untreated cross-ties there was a crumbling of the fibers close to the spike, while in the untreated cross-ties the fibers were torn out in deep slivers extending from the spike to the supporting blocks on the testing machine.

It is thought that a withdrawal of $\frac{1}{4}$ inch should not be exceeded in practice, and that for practical purposes in comparing the holding power of spikes and cross-ties under different conditions, either the resistance for a withdrawal of $\frac{1}{4}$ inch, or the maximum resistance, in case it occurs for a smaller withdrawal should be taken. Since the maximum resistances of chestnut and loblolly pine (untreated) occur at $\frac{3}{16}$-inch withdrawal, these values should then be compared with that of white oak for a withdrawal of $\frac{1}{4}$ inch, in which case the respective percentages become 131 and 85, respectively. Three distinct kinds of preserving solutions were used for the cross-ties, viz.: creosote, zinc creosote, and zinc tannin. A study of the results fails to show such marked differences in the holding power of spikes in the treated cross-ties as to warrant any definite statement regarding the relative effect of these solutions.

Only three sizes of spikes were used in the tests, the thicknesses being $\frac{9}{16}$, $\frac{19}{32}$, and $\frac{5}{8}$ inch in diameter, respectively, the corresponding number tested being 72, 36, and 102. In the black oak, red oak, beech, and sweet gum the resistance of the $\frac{5}{8}$-inch spike is larger than that of the $\frac{9}{16}$-inch spike, but in white oak and water oak the reverse is true. In white oak and water oak the resistance of the $\frac{9}{32}$-inch spike is less than that of either the $\frac{9}{16}$- or $\frac{5}{8}$-inch spike, while in black oak and beech the relation is reversed. The holding power varies directly with the penetration, not counting the taper point.

Three types of tapered points were employed, the ordinary blunt wedge, the chisel or sharp wedge, and the bevel point

made by two bevel cuts on the wedge. The general averages for the tests in which eight kinds of timber were used, indicate a superiority of the blunt and bevel points over the chisel points of 12 and 14 percent, respectively, for a withdrawal of $\frac{1}{4}$ inch, and of 5 and 3 percent for the maximum resistance. The blunt-pointed spike developed the highest resistance for withdrawals of $\frac{1}{8}$ and $\frac{1}{4}$ inch in a larger number of tests than the bevel-pointed one, and hence may be regarded practically as the most efficient.

It is stated in the bulletin from which these facts are taken that "an examination of the tie showed that the blunt-pointed spike disturbed more fiber than either the chisel or the bevel-pointed spikes, the last two disturbing about the same amount. The examination also showed that the blunt-pointed spike tore rather than cut the fibers, and deposited them in unequal bundles along its faces, while the chisel-pointed spike cut the fibers and deposited them quite uniformly both across and in front of each face. The bevel-pointed spike forced a majority of the fibers to the front face and toward the corners. The relatively high holding power of both the blunt and the bevel-pointed spikes is due to this unequal concentration of the fibers."

A set of tests was made to study the effect of boring holes for the spike. In some cases the hole was $\frac{1}{16}$ inch less, and in other cases $\frac{1}{8}$ inch less in diameter than the thickness of the spike. The depth of boring was less than the depth of penetration of the spike, so as to force its point into the undisturbed fibers of the wood. In the majority of cases the spike driven into a bored hole develops a larger resistance than when driven in the ordinary way, for a withdrawal of $\frac{1}{4}$ inch or less, but gives a smaller maximum resistance.

It was also found that the holding power of a spike which was withdrawn a moderate distance as in practice, and then redriven, was only from 61 to 82 percent of the original value, 36 tests being made in six kinds of wood.

The holding power of the screw spike is always greater than that of the ordinary spike. For a withdrawal of $\frac{1}{4}$ inch in the hard woods the resistance of the screw spike is from 160 to 221 percent of that of the ordinary spike, while in the soft woods the range is from 117 to 258 percent. The average gain in the hard woods is 76 percent and in the soft woods 98 percent. The holding power of the screw spike for a penetration of 5 inches and a withwrawal of $\frac{1}{4}$ inch for different kinds of wood is as follows: water oak, 9180; black oak, 10 420; red oak, 10 400; white oak, 11 900; ash, 10 470; beech, 13 140; elm, 10 090; poplar, 6210; chestnut, 6340; sweet gum, 7710; and loblolly pine, 9050 pounds.

A comparatively small number of tests of railroad spikes have been made at the Watertown Arsenal the results of which are recorded in Tests of Metals, etc., 1884, page 448; 1887, page 926; and 1889, page 595. The spikes were $\frac{9}{16}$ and $\frac{5}{8}$ inch square and mostly $5\frac{1}{2}$ inches long, the depth of penetration varying from 3 to $5\frac{1}{4}$ inches. Some of the cross-ties were new, and others had been in service in the track. The holding power is expressed in pounds per square inch of the surface of contact between the wood and spike exclusive of its tapered end. The average maximum resistances in different woods are as follows: white oak, 6 tests, 570; hemlock, 4 tests, 390; yellow pine, 11 tests, 359; chestnut, 10 tests, 341; white pine, 4 tests, 262; and white cedar, 4 tests, 186 pounds per square inch. See also an article by JAMES E. HOWARD, in Railroad Gazette, May 1, 1891, vol. 23, page 298, which discusses the results recorded in the second reference given at the beginning of this paragraph.

A series of still older experiments by A. M. WELLINGTON, in which ten species of wood, both treated and untreated, were employed, are described in an article in Railroad Gazette, Dec. 17, 1880, vol. 12, page 668. Another set of tests was made by A. J. Cox in 1891 in the engineering laboratory of the University

of Iowa, to investigate the comparative holding power of the common and two other forms of railroad spikes. The results were originally published in the technical college journal called The Transit, and are reprinted in Engineering Record, vol. 24, page 385, Nov. 14, 1891.

An excellent half-tone illustration of the destructive effect upon the fibers of a cross-tie when a spike is driven in the ordinary manner, and showing the comparative condition when a hole is previously bored, may be seen in Railroad Gazette, Feb. 2, 1894, vol. 26, page 81. A similar illustration on a smaller scale is also given in the University of Illinois Bulletin, previously referred to in this article.

Prob. 7. Refer to University of Illinois Bulletin No. 18, plates II and III, and compare the work done in extracting an ordinary railroad spike from a chestnut and from a white oak cross-tie.

ART. 8. LATERAL RESISTANCE OF SPIKES.

On railroad curves the track spikes which hold the outer rail in position are subjected to lateral pressure due to the centrifugal force of the moving train, and to other forces caused by the tendency of locomotive or car trucks to continue in a straight line. In case tie plates with shoulders are used the lateral pressure is resisted by the spikes on both sides of the outer rail, but when no tie plates are employed, it is resisted only by the spikes on the outside of the rail. In either case the friction between base of rail and cross-tie helps to resist the lateral pressure.

The horizontal force applied at the edge of the rail flange causes a compression of the fibers of the wood on the opposite side of the spike above and on the same side below as indicated in Fig. 8*a*. The maximum compression occurs at the surface of the cross-tie, and since the deflection of the spike is comparatively slight for safe working conditions, the pressure on the

wood may be regarded to vary the same as the ordinates to a straight line.

To deduce a formula expressing the relation between the rail pressure P and the maximum unit compression S on the fibers of the cross-tie, let e be the distance from the bearing of the rail flange in the spike to the surface of the cross-tie, l the penetration of the spike in the cross-tie, and let a and c be the depths of the wood surface in compression on opposite sides of the spike (Fig. 8*b*). The resultants P' and P'' are proportional to the

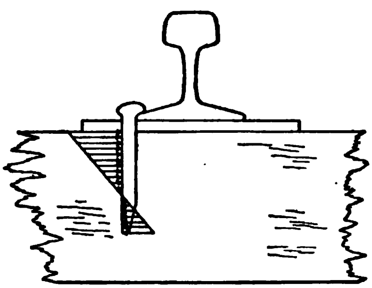


FIG. 8*a*.

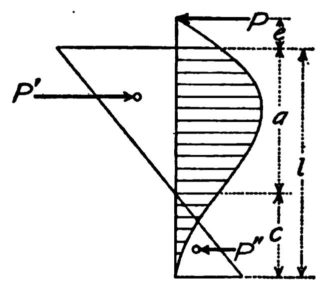


FIG. 8*b*.

two unshaded triangular areas, respectively, or $P' : P'' = a^2 : c^2$, whence $P' = P''a^2/c^2$. As the sum of the horizontal forces must equal zero, there follows the relation $P' = P + P''$. The diagram also shows that $l = a + c$. Taking moments about some point on the line of action of P,

$$P''(l + e - \tfrac{1}{3}c) - P'(\tfrac{1}{3}a + e) = 0.$$

Substituting the value of P' given above, replacing a by $l - c$, and reducing, there follows

$$c = \frac{l(l + 3e)}{3(l + 2e)} \text{ and } a = l - c = \frac{l(2l + 3e)}{3(l + 2e)}. \qquad (1)$$

Designating the width of the spike by b, the resultant P' equals $\frac{1}{2}abS$, and remembering that $P = P' - P''$, there is obtained

$$P = Sbl^2/(4l + 6e), \text{ or } S = P(4l + 6e)/bl^2. \qquad (2)$$

For example, let the spike be $\frac{5}{8}$ inch square in section and 6 inches long over all, or $5\frac{1}{2}$ inches below the head. Let the tie plate be $\frac{1}{2}$ inch thick and the edge of the rail flange $\frac{1}{4}$ inch thick; then $b = \frac{5}{8}$ inch, $l = 4\frac{3}{4}$ inches, and $e = \frac{5}{8}$ inch. Substituting these values in the preceding formulas, the following values are obtained: $a = 3.002$ inches, $c = 1.748$ inches, $P = 0.620\ S$, and $S = 1.613\ P$. For a working unit stress on the ends of the fibers of 1200 pounds per square inch, the lateral resistance of one spike is $P = 0.620 \times 1200 = 744$ pounds. The ordinates to the curved line in Fig. 8*b* represent the bending moments in the spike. The section where the shear is zero is 1.25 inches below the surface of the cross-tie, and hence the maximum bending moment is $M = 744 \times 1.875 - 744 \times 0.680 = 889$ pound-inches. The lever arm 0.680 inch in this equation is the distance to the section from the center of gravity of the trapezoidal load area above the section as indicated in Fig. 8*b*. Equating this value of the bending moment to the resisting moment of the spike, the unit stress in the outer fiber is found to be 21 800 pounds per square inch.

For a practical application of these formulas see an article on Stresses in Track Fastenings, by W. C. CUSHING, including one by E. E. STETSON on A Study of Rail Pressures and Stresses in Track Produced by Different Types of Steam Locomotives when rounding Various Degree Curves at Different Speeds, published in Proceedings of the American Railway Engineering and Maintenance of Way Association, 1909, vol. 10, page 1431.

Numerous impact tests to determine the relative efficiency of ordinary railroad and screw spikes to resist lateral displacement were made at the University of Illinois Engineering Experiment Station, a record of which was published in 1906 in the second part of Bulletin No. 6, by R. J. WEBBER. The results show that the screw spike has a higher efficiency than the $\frac{9}{16}$-inch ordinary

spike in all but two of the eleven kinds of wood used, and higher than the $\frac{5}{8}$-inch spike in all but three kinds of wood. The screw spikes had a core diameter of either $\frac{11}{16}$ or $\frac{21}{32}$ inch.

Some tests were also made by WALKER and CROSS on common wire spikes as well as boat spikes when driven through a cleat into a block, and then tested in the manner described for nails in the preceding article. See Journal Association of Engineering Societies, vol. 19, page 260, December, 1897.

Prob. 8. Find the lateral resistance of a $\frac{9}{16}$-inch spike, 5 inches long under the head and without a tie plate under the rail, the allowable compression in the cross-tie being 1000 pounds per square inch. Compute also the unit-stress in the outer fiber of the spike.

ART. 9. DRIFT BOLTS.

A drift bolt is a piece of round or square iron or steel, of specified length, with or without head or point, driven as a spike. Different forms of head and point are shown in Fig. 9*a*. Before driving the bolt a hole must be bored somewhat smaller than the drift bolt. The best relation between diameters of bolt and hole is discussed in the next article.

A common form of drift bolt differs but little from a boat spike, except in length. A drift bolt acts like a nail, not only preventing lateral movement of the parts connected, but also their separation in a direction parallel to its axis. A dowel only prevents lateral displacement and is usually thicker and shorter than a drift bolt. Drift bolts should be long enough to give a good hold in the last timber which they are to penetrate.

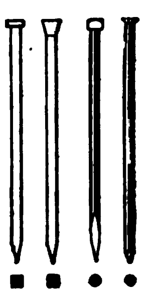


FIG. 9*a*. Drift Bolts.

Fig. 38*g* in Art. 38 illustrates the use of drift bolts in fastening the cap to the piles of a trestle bent, and in Fig. 73*b* in Art. 73 they are used to fasten the caps to the posts, as well as the sill to the piles.

They are designated spikes on the latter drawing. In one-half of the bent which has brick footings, drift bolts are also employed to fasten the sill to the posts, while in the other half dowels are used for this purpose.

In its report in 1892 on various methods of framing trestle bents a Committee of the American Association of Railway Superintendents of Bridges and Buildings made the following reference to drift bolts: "The third method used is by means of drift bolts, which act exactly as a nail or spike. They not only prevent any lateral movements in the timbers, but prevent the contiguous faces from drawing apart, by the friction of the wood on their sides. Drift bolts are very much the simplest and least expensive of the various types of fastenings used, and were it not for the same disadvantages (see Art. 14) as found in the use of dowels, they would be more generally used. However, this method is very common, and for all temporary work it is the best and most rigid manner of securing a strong joint at so slight a cost."

In the discussion of that report attention was called to the fact that large heads on drift bolts are objectionable since they crush the fibers on being driven into the wood, and in the cavities so formed the rain water accumulates and causes decay; and that the difficulty of relining a stringer which is so fastened to a cap is materially increased. It was stated that by using drift bolts with chisel points and no heads, and using augurs of smaller diameter than the bolt, that sufficient head is formed in driving to answer all purposes, while facilitating the economic relining or renewal of any part. In some standard designs the head of the drift bolt is driven $\frac{3}{4}$ inch below the surface and the cavity filled with pitch to prevent the entrance of water. When drift bolts are used in trestle construction, their diameter is usually $\frac{3}{4}$ inch and sometimes $\frac{7}{8}$ inch.

Figs. 9*b*, *c*, and *d* show the use of drift bolts in fastening the

timbers of the sides and roof of a pneumatic caisson for a building foundation. They are $\frac{3}{4}$ inch in diameter and 30 inches long. Fig. 9e indicates how the various courses of timber in a dam are connected by drift bolts. In the solid walls of 12 by 12

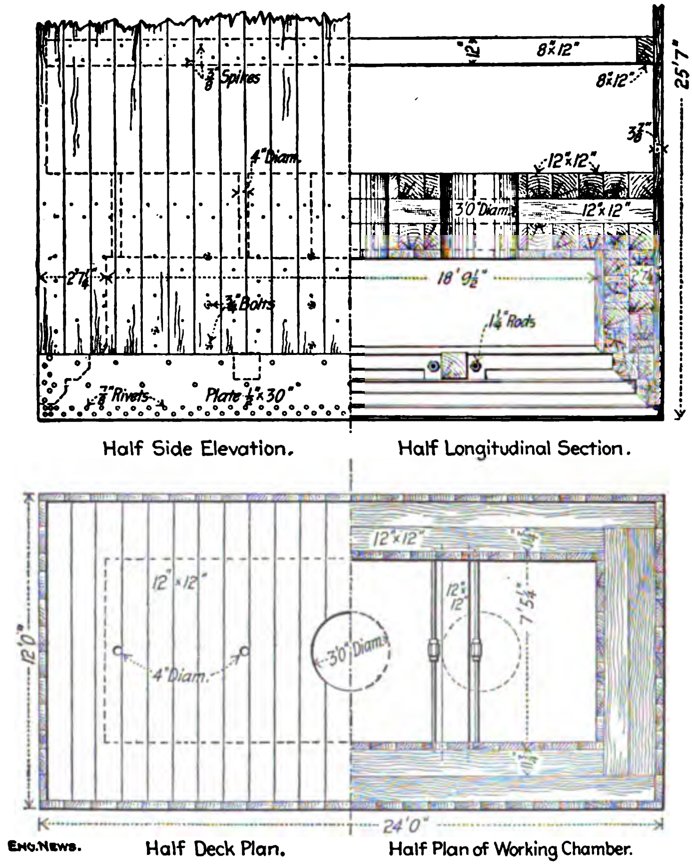
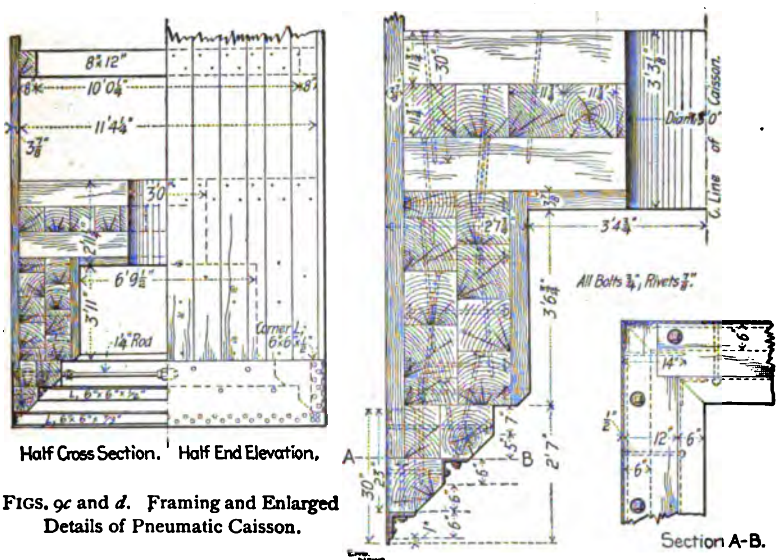


FIG. 9b. Pneumatic Caisson for Foundations of Gillender Building, New York City. See Engineering News, vol. 37, page 13, Jan. 7, 1897.

inch timbers of the cribs above the caissons of the Ohio River bridge at Cairo, Ill., the drift bolts were $\frac{7}{8}$ inch square and 30 inches long, those driven into each course of timber being

spaced 3 feet apart. The largest crib was 30 feet wide and 70 feet long. In the large open cribs of the Poughkeepsie bridge foundations, 1-inch drift bolts 30 inches long were used, and spaced about $1\frac{1}{2}$ feet apart. The largest crib was 60 by 100



FIGS. 9c and d. Framing and Enlarged Details of Pneumatic Caisson.

feet, with six transverse and one longitudinal dividing walls between the compartments. In the construction of cribs it is in most cases the best plan simply to lap the timbers at the corners as well as elsewhere, and depend upon the drift bolts to hold them in place. The use of drift bolts in the construction of a dam, consisting of an open crib work filled with rock and sheeted over, may be seen in an article on Lock No. 3, Cumberland River, in Engineering News, vol. 61, page 504, May 6, 1909.

The plan of simply lapping the large timbers of cribs and caissons and depending upon the drift bolts to hold them in place has been strongly commended by experienced engineers.

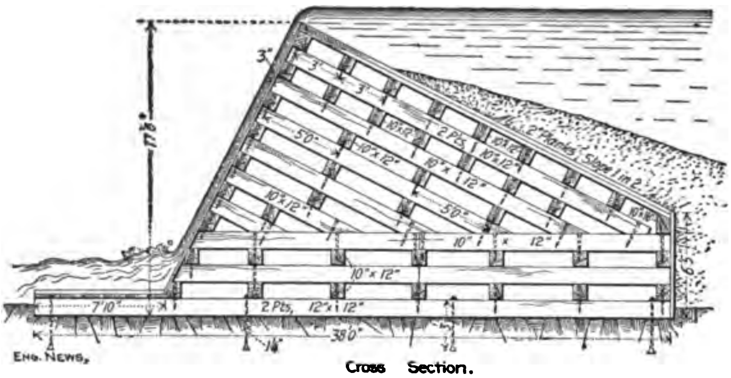


FIG. 9c. Cross-section of Bear River Dam. See Engineering News, vol. 35, page 84, Feb. 6, 1896.

Prob. 9. Refer to Proceedings American Railway Engineering and Maintenance of Way Association, 1905, vol. 6, page 43, and one of the folding plates accompanying the article by A. F. ROBINSON. Make a drawing of the two drift bolts shown to one-half size and add the note of instruction.

ART. 10. HOLDING POWER OF DRIFT BOLTS.

The most systematic investigation of the adhesive power of round drift bolts was made by J. B. TSCHARNER in the engineering laboratory of the University of Illinois. The results were originally published in the Technograph, a local technical magazine, 1889–1890, No. 4, page 53, and a brief summary was given in Engineering News, vol. 25, page 199, Feb. 28, 1891. Rods or drift bolts of steel 1 inch in diameter were used, the diameters of the holes bored were $\frac{15}{16}$, $\frac{14}{16}$, $\frac{13}{16}$, and $\frac{12}{16}$ inch, respectively, in both yellow pine and white oak timber, and the total number of tests was 126. The ends were not pointed, but the sharp edges were hammered down so as not to cut the timber.

The results of 45 tests, in which the rods were driven from 3 to 12 inches into yellow pine, with holes $\frac{15}{16}$ inch in diameter, prove that the adhesion is directly proportional to the depth to which the bolt is driven.

ULTIMATE HOLDING POWER PER LINEAR INCH OF 1-INCH ROUND DRIFT BOLTS.

Diameter of hole	15/16	14/16	13/16	12/16 inch
White oak	1300	1778	2500	1133 pounds
Yellow pine	375	633	788	400 pounds
Ratio	3.5	2.8	3.2	2.8
White oak	52	71	100	45 percent
Yellow pine	48	80	100	51 percent

The preceding table indicates that the holding power in white oak is approximately three times as great as in yellow pine. It also shows that the holding power for the $\frac{13}{16}$-inch holes is greater than for any of the others in both kinds of wood. As the holding power diminishes very rapidly for the next smaller size of hole, it is apparent that the elastic limit of the wooden fibers is passed. Therefore to develop the maximum holding power of the bolt the diameter of the hole should be about 80 percent of that of the bolt, or the section area of the hole should be about 66 percent of that of the bolt.

The results given in the following table are the average values for 6 tests in yellow pine and 3 tests in white oak for every size of hole. The drift bolts were driven to a depth of 6 inches in every case.

RELATIVE HOLDING POWER OF DRIFT BOLTS WHEN PERPENDICULAR AND PARALLEL TO THE FIBERS.

Diameter of hole	15/16	14/16	13/16	12/16 inch
Yellow pine:				
Perpendicular	375	633	788	400 pounds
Parallel	200	280	344	222 pounds
Ratio	1.9	2.3	2.3	1.8
White oak:				
Perpendicular	1300	1778	2500	1133 pounds
Parallel	617	817	1033	867 pounds
Ratio	2.1	2.2	2.4	1.3

The data show that the holding power for drift bolts driven perpendicular to the fibers is approximately double that when

driven parallel to the fibers. The average ratio is practically the same for both kinds of wood.

It was also found that in pulling a drift bolt the holding power decreases very rapidly as soon as motion begins, and that if originally driven 6 inches into the timber, only about 20 percent of the initial resistance remains when it is withdrawn 1 inch, after which it decreases directly with the distance drawn out, during the following year.

The tests described above were continued by J. H. POWELL and A. E. HARVEY, who used 1-inch square drift bolts and yellow pine timber. Five tests were made for each size of hole, and the bolts were also driven to a depth of 6 inches. The average holding power per linear inch was found to be as follows: for a 1-inch hole, 662 pounds; for a $\frac{15}{16}$-inch hole, 710 pounds; for a $\frac{14}{16}$-inch hole, 777 pounds; and for a $\frac{13}{16}$-inch hole, 675 pounds. The largest value is given by the $\frac{14}{16}$-inch hole, which has a section area equal to 60 percent of that of the bolt, or nearly the same ratio as for round bolts.

After the bolts were withdrawn and the timbers were split open to examine the condition of the wood around the holes, it was found that in those larger than $\frac{14}{16}$ inch, only the corners of the bolts had held effectively, while in the smaller holes the wood fibers were crushed and torn.

Although the section area of the 1-inch square drift bolt is 33 percent larger than that of the 1-inch round bolt, its maximum holding power is practically about the same. It must therefore be concluded that round drift bolts are more economical than square ones.

A series of 43 tests was made at the Watertown Arsenal, the results of which are recorded in Tests of Metals, 1902, page 577. The diameters of the bolts are 1, $\frac{7}{8}$, $\frac{3}{4}$, $\frac{5}{8}$, and $\frac{1}{2}$ inch, respectively. In the Douglas fir wood five sizes of holes were bored for each size of bolt, the largest diameter in each case being $\frac{1}{16}$ inch less

than that of the bolt, the other holes decreasing successively by $\frac{1}{16}$ inch. In the white oak only 3 or 4 sizes of holes were bored for each size of bolt. The largest adhesive resistance for each size of bolt is given in the following table. The resistance per square inch relates to the surface of adhesion between the bolt and the timber.

ULTIMATE HOLDING POWER OF ROUND DRIFT BOLTS.

Diameter of bolt	1	7/8	3/4	5/8	1/2 inch
In Douglas fir:					
Diameter of hole	13/16	5/8	1/2	3/8	3/16 inch
Resistance per linear inch	2250	1901	1566	1555	1185 pounds
Resistance per square inch	724	696	656	762	741 pounds
In white oak:					
Diameter of hole	13/16	11/16	5/8	7/16	3/16 inch
Resistance per linear inch	3250	2588	1950	2462	2114 pounds
Resistance per square inch	1050	947	816	1210	1320 pounds

The averages of the resistance per square inch given above are 716 pounds for Douglas fir and 1068 pounds for white oak, the ratio being 1.49. The corresponding averages, when all sizes of holes are considered, are 539 pounds for Douglas fir and 957 for white oak, the ratio being 1.78. These values are respectively 73.0 and 74.3 per cent of the general averages for the lag screws tested at Watertown under the same conditions (see Art. 13). The resistance per square inch in the white oak is 1030 pounds for the 1-inch bolt in a $\frac{7}{8}$-inch hole, 947 pounds for the $\frac{7}{8}$-inch bolt in a $\frac{3}{4}$-inch hole, 789 pounds for the $\frac{3}{4}$-inch bolt in a $\frac{9}{16}$-inch hole, 1200 pounds for the $\frac{5}{8}$-inch bolt in a $\frac{1}{2}$-inch hole, and 1280 pounds for the $\frac{1}{2}$-inch bolt in a $\frac{1}{4}$-inch hole; thus showing that a variation of $\frac{1}{16}$ inch in the size of the hole reduces the adhesion but slightly from the maximum value for each bolt.

By comparing the resistance per linear inch for the 1-inch bolt in Douglas fir with the corresponding resistance given in the first table in this article, it is evident that the yellow pine used was not the Southern longleaf species.

The following average results are computed from the record of experiments made on drift bolts at the Watertown Arsenal given in Tests of Metals, etc., 1889, page 595. The sizes of bolts tested are $\frac{1}{2}$, $\frac{5}{8}$, and $\frac{3}{4}$ inch in diameter, and the depth to which they were driven varies from 4 to $5\frac{1}{4}$ inches. The maximum holding power for each kind of timber is given in pounds per square inch of surface of contact between bolt and wood. The average holding power for 11 tests in cedar is 143; for 8 tests in chestnut is 365; for 2 tests in yellow pine is 397; and for 5 tests in white oak is 795 pounds per square inch. A few tests were also made which indicate that the resistance is diminished about two-thirds by lubricating the wood with block oil or coating the bolt with asphalt varnish or with mastic and white zinc.

An extensive series of tests was made in 1874–1877 by A. NOBLE and C. P. GILBERT, U. S. Assistant Engineers, the results of which are published in the Report of the Chief of Engineers, U.S.A., 1884, page 2051. The total number of tests made was about 500, but the results indicate a considerable range both for the same stick and in different sticks of white pine used at both locations where the tests were made. For example, the holding power for a 1-inch round drift bolt in a $\frac{3}{4}$-inch hole in 8 different sticks varied from 731 to 1108 pounds per linear inch, each value being the mean of from 4 to 6 tests, while the range for single tests was still greater. In a few cases the highest value for one stick was more than double the lowest one for the same stick.

In general, the same relations between sizes of holes and bolts and between round and square drift bolts were obtained as those previously described in this article. The following table gives the average ultimate holding power per linear inch for 1-inch round drift bolts in white pine when tested as soon as driven. The respective means of over 150 tests of different conditions

HOLDING POWER PER LINEAR INCH OF 1-INCH ROUND DRIFT BOLTS IN WHITE PINE.

Diameter of hole	11/16	12/16	13/16	14/16 inches
Holding power	837	856	899	792 pounds
Number of tests	18	42	25	5

indicate that the resistance is about 10 percent greater when tested 7 months after driving. It was also found that a hole $\frac{14}{16}$ inch in diameter developed the greatest holding power for a 1-inch square drift bolt, the mean value for 48 tests being 901 pounds per linear inch. Some experiments were made on $1\frac{1}{8}$- and $\frac{3}{4}$-inch round and $\frac{5}{8}$-inch square drift bolts which show that practically the same relations hold between the section areas of the hole and bolt as those previously given.

The holding power for different sizes of drift bolts in timber of the same quality is closely proportional to the surface of contact between the bolt and the timber. Since the principal part of the resistance is due to the bearing of the side of the bolt on the ends of the wooden fibers, the holding power in different kinds of timber is approximately proportional to the compressive strength of short blocks for straight-grained clear wood.

The mean of 150 tests under various conditions shows that it requires only about 60 percent of the power to pull the drift bolt through the timber in the direction in which it was driven as to withdraw it in the opposite direction. It is also found that smooth rods have a greater holding power than ragged ones when pulled in either direction. The resistance is reduced a little more than 25 percent by moderate ragging, and over 50 percent by excessive ragging, these values being the average of 50 tests. When the rods were threaded, however, as in lag screws, the resistance to withdrawal is increased over 50 percent.

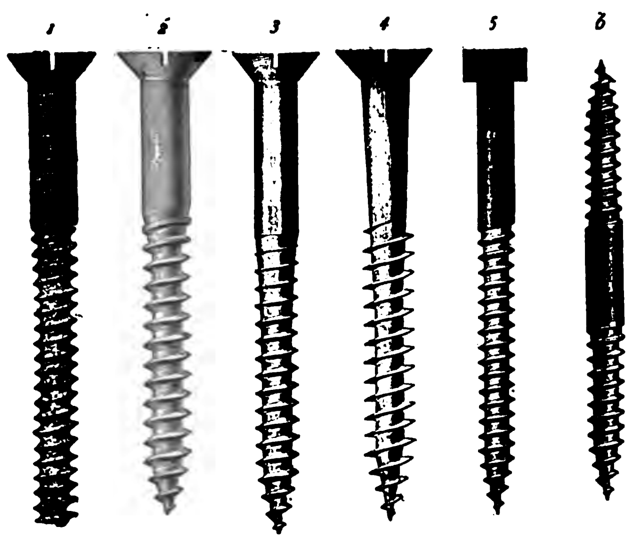
In some experiments made during the construction of the foundations of the Brooklyn bridge, it was found that a 1-inch round rod driven into a $\frac{7}{8}$-inch hole in longleaf yellow pine

developed a holding power of 1250 per linear inch. When the wood was very heavy and contained more pitch, the resistance was about 10 percent greater, but in lighter wood which contained less pitch it was about 20 percent less. In similar experiments at the Poughkeepsie bridge, the holding power of 1-inch round drift bolts driven into a $\frac{7}{8}$-inch hole in white pine and in Norway pine was 830 pounds per linear inch.

Prob. 10. With the aid of a diagram showing sections of drift bolts, and of the holes bored in the timber, as well as the direction of the fibers, it is required to give a rational explanation why the holding power of a square drift bolt is less than that of a round one of the same section area.

ART. 11. WOOD SCREWS.

A common wood screw is a metal fastening used to attach either a piece of wood or metal to a timber into which the screw



FIGS. 11a-f. Four Wood Screws, Coach Screw and Dowel Screw.

penetrates. As illustrated in Fig. 11c it contains a slotted head, a smooth shank, and a thread attached to a core which termi-

nates in a point. The core is tapered so that its pressure on the sides of the hole increases as it is driven deeper.

Screws are used instead of nails when greater holding power is required, to avoid the danger of splitting or other injury, to secure better fitting, or to separate the parts readily when this may be desired afterwards. Screws are classified according to their use, the shape of the head, the form of the thread, or according to some special feature of their construction. The flat head is used when the material is thick enough to allow the head to be countersunk in it, while other forms are used either for thinner material or for ornamental purposes or both.

Figures 11*a–d* illustrate the progress made in the manufacture of wood screws. No. 1 is from an old sample card imported by JONATHAN CONGDON of Providence; No. 2 was invented by THOMAS SLOAN in 1846; while Nos. 3 and 4 are the invention of CHARLES D. ROGERS in 1876 and 1892, respectively. In the forms of 1846 and 1876 the thread is cut on the body of the screw blank, but that of 1892 is produced by a cold forging process.

The advantages claimed for the form invented in 1892 are that it has a forged nick, a point that is exactly centered, and a metallic skin over the entire surface, making it stronger than a cut screw; that it requires only one size of hole to be bored, less power to drive it in, and has greater holding power. However, the cut screw of 1876 still remains in general commercial use, notwithstanding the decided superiority of the later design which is produced by cold forging.

Wood screws were in use long before 1760, when a patent was issued for cutting screws by machinery, having previously been made entirely by hand. In 1832 hand threaders were still in use in Providence, R.I., and the use of machinery for manufacturing screws was limited. In 1854 a greatly improved machine was invented in the United States which revolutionized screw-

making. In about 1870 steel was first used for commercial wood screws, and by 1890, or very soon thereafter, the use of iron for this purpose was practically discontinued.

Screws are made in various lengths up to 6 inches, and some lengths are made in as many as 18 different diameters. The gage numbers ranging from zero to thirty refer to the American screw gage which differs from the ordinary wire gage. The relation between the diameter of the shank and the screw gage is given by the equation $D = 0.0578 + 0.01316\ N$, in which D is the diameter in inches, and N the number of the screw gage. Screws are made of iron, steel, brass, copper, and other materials, with different kinds of finish or plating. In damp places brass screws should be used. Tables giving the available sizes may be found in the catalogues of screw manufacturers, or of wholesale hardware dealers.

In good practice a hole is bored of the diameter of the core before inserting the screw. In hard wood a second hole is required of the diameter and depth of the shank to keep the wood from splitting or the screw from twisting off.

The helicoid-shank wood screw has been devised to avoid the necessity in hard wood and fine work of boring the second hole for the shank of the common form of screw. In this improved type (Fig. 11*g*), the shank has a series of helical V-shaped ribs or threads of very large pitch which serve to ream out the hole to an exact fit during the operation of turning the screw.

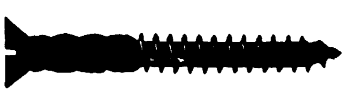
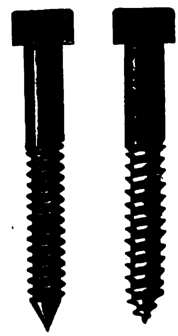


FIG. 11*g*. Helicoid-shank Screw.

When a common screw is partially driven into the wood with a hammer before using the screw-driver to drive it home, the fibers are seriously injured, and the holding power of the screw materially reduced. A cold-forged drive screw has been invented which, owing to its peculiar form of thread of large pitch, turns like a screw when driven with a hammer, and does not break

the fibers of the wood. Its holding power equals that of a common screw, as frequently inserted, while its cost is less.

Lag screws and coach screws are large screws with square heads like that of a bolt and are turned with a wrench. They are used in heavy timber construction. Lag screws have cone points, as in Fig. 11*h*; while coach screws have gimlet points, as in Figs. 11*e* and *i*. The latter illustration also shows the rachet thread which is sometimes used on either form of screw. A hole must first be bored of the same diameter and depth as the shank, and then continued with a diameter equal to that at the root of the thread. The diameters range from ¼ to 1 inch, and the lengths from 1½ to 12 inches. See catalogues of the manufacturers.



FIGS. 11*h* and *i*.

The screw spike has come into use in this country since 1906 to fasten track rails to the cross-ties on elevated railroads and on ordinary ballasted track, in a more effective manner than by means of the common railroad spike which so seriously injures the fibers of the wood, thus reducing its strength and promoting decay. In Arts. 7 and 8 references are given to articles and pamphlets in which their proportions and strength are given, and a history of their extensive use in Europe.

Prob. 11. By means of a screw manufacturer's catalogue, prepare a table giving the lengths of lag screws for the different diameters.

ART. 12. HOLDING POWER OF COMMON SCREWS.

An extensive investigation of the holding power of ordinary cut wood screws was made in the laboratory of the College of Civil Engineering, Cornell University, by N. M. WORKS and W. J. GRAVES in 1897 and 1898. The tests were made with white pine, yellow pine, and white oak, the screws being furnished by the American Screw Company.

The investigation included 10 lengths of screws from 1 to 6 inches, and 8 diameters from No. 4 to No. 30, or 0.1105 to 0.4520 inch, the diameter at the root of the thread ranging from 0.070 to 0.235 inch. Three to six different diameters of screws were tested for each length, while from 3 to 5 specimens were tested for each size and each kind of timber, making 503 tests, in addition to several hundred preliminary ones to determine the proper relations of diameter of hole to that of screw, the effect of driving part way with a hammer, and other conditions.

It was found that except in soft wood and for very small sizes of screws the holding power is increased by boring a hole before inserting the screw. In white pine the best results were obtained by varying the diameter of the hole from 82 to 100 percent of the diameter of the inner cylinder or core of the screw, in yellow pine from 90 to 100, and in white oak from 85 to 105 percent. The ratio increases as the diameter of the screw decreases. It also increases with the hardness of the wood. Practically the same relations were obtained with the screws parallel to the fibers of the wood as when perpendicular to the fibers. The depth of the hole should extend to the section where the diameter of the threaded portion diminishes rapidly as it approaches the point. Individual tests with holes of the same ratio to the core of the screw and in the same stick of timber give a variation in holding power far greater than that of the average holding power for different-sized holes between the limits given above. This fact indicates that lack of homogeneity in timber influences holding power far more than the given changes in the size of hole. It will therefore be sufficiently precise for most purposes to make the diameter of the hole equal to that of the core of the screw.

Tests for the proper size of hole for the shank of a screw indicate that in white pine its diameter should be about 77 percent of that of the shank, and in the harder woods about 83

percent. However, since the holding power due to the friction of the shank averages only about 5 percent of the total, and it is secured by developing a tendency to split the timber, it is usually best to bore the hole of the same diameter as the shank, and to a depth slightly less than that where the thread begins when the screw is in place.

In longitudinal sections of timbers after screws, inserted perpendicular to the fiber, are partly pulled out, inspection shows that the fibers are bent up near the hole and pulled apart and damaged for a distance of about $\frac{3}{4}$ of an inch each way in white pine, and 1 inch each way in the harder woods.

Although ordinary observation of a cross-section of the timber fails to detect any damage to the grain beyond the reach of the thread, if the spacing of the screws is too small, their holding power is reduced. It is desirable therefore to space the larger screws not less than 1 inch in a direction across the grain, and not less than 2 inches along the grain. In hard woods the corresponding spacing should not be less than $1\frac{1}{2}$ and $2\frac{1}{2}$ inches respectively. When the screws are inserted parallel to the fibers, the damage due to pulling them out is mostly local, and hence the smaller spacing given for each kind of timber will be sufficient.

The effect of lubrication, as demonstrated by tests in white pine, is a marked reduction in the work needed to insert them and a small increase in holding power. The increase is greater in the harder woods where lubrication becomes a necessity, especially with the larger screws. The best lubricant is the heaviest one which will adhere to the threads. Tar soap gave the best results in these tests.

Driving the screws part way with a hammer, as is frequently done in practice, always lessens the holding power, either by damaging the fibers or the screws.

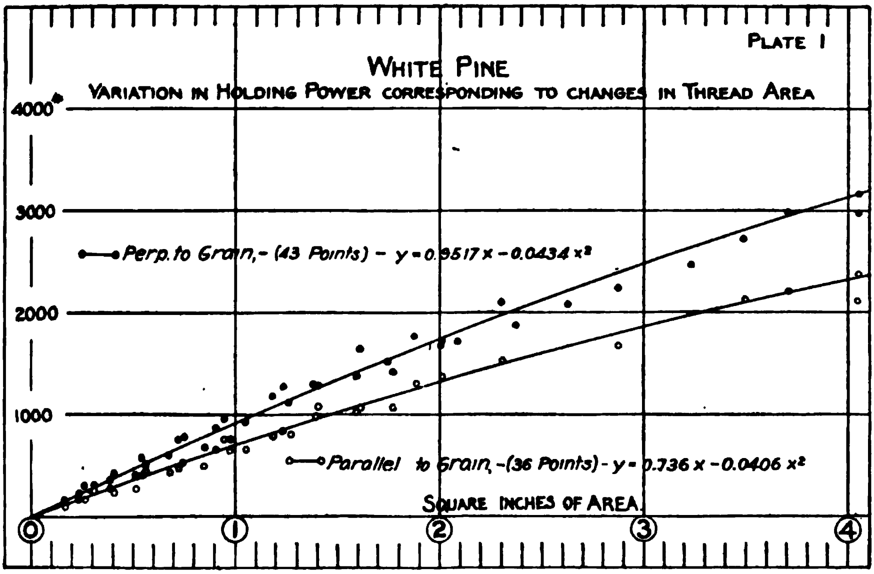


FIG. 12*a*. Holding Power of Wood Screws in White Pine.

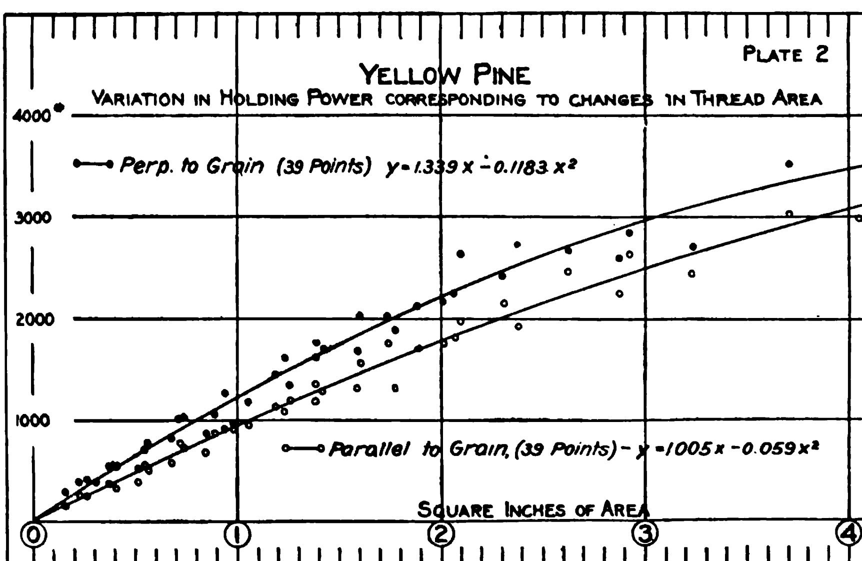


FIG. 12*b*. Holding Power of Wood Screws in Yellow Pine.

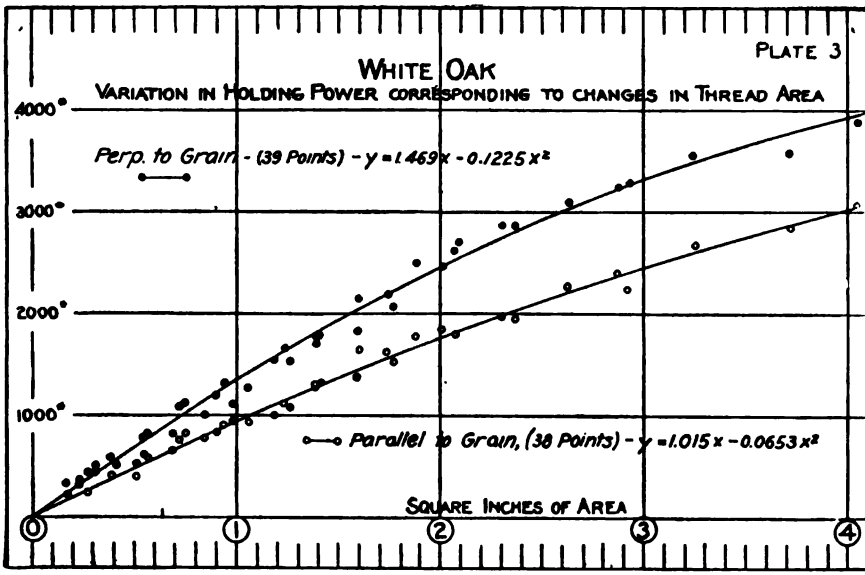


FIG. 12c. Holding Power of Wood Screws in White Oak.

The holding power for screws inserted both perpendicular to and parallel to the fibers is shown graphically in Figs. 12*a*, *b*, and *c*. Each dot or circle represents the average value for one size of screw, and the equation of each line gives the relation between the holding power and the net shearing area, called the thread area on the diagrams. This area is the product of the effective length of the threaded portion and the outer circumference at the middle of the effective length. The effective length of the threaded portion of the screw excludes from 3 turns for No. 4 to 1½ turns for No. 28 to allow for the tapering end. The relation between holding power and shearing area is expressed by a parabola in each case, the curvature being less for the soft than for the hard woods. For screws with thread areas less than two square inches which includes nearly all screws less than four inches long, the relation may be expressed with sufficient precision by a straight line, making

the holding power directly proportional to the shearing area. Including all sizes and lengths of screws tested, the average resistance in pounds per square inch of shearing surface or thread area is 905 in white pine, 1199 in yellow pine, and 1304 in white oak, the screws being inserted perpendicular to the fibers.

The average ratio of the holding power for screws parallel to the fibers, to that perpendicular to the fibers, was found to be 74 percent for white pine, 77 percent for yellow pine, and 78 percent for white oak. The average holding power in white pine is 72.5 percent of that in yellow pine and 69 percent of that in white oak, while the corresponding relation between yellow pine and white oak is 93 percent.

The holding power per pound is greater for the smaller sizes of screws. For screws of a given length it decreases as the diameters increase, and for those of a given diameter there is a slight increase with the length. The proportions of screws are an important factor in their holding power and depend upon the quality of the timber in which they are used. In long screws with the smaller diameters the heads pull off before the fibers of the wood fail, if hard wood is used, which indicates that such sizes should be employed in softer wood. For the kinds of timbers used in these tests the economical limit of length is about four inches, and with this length the economical use of lag screws begins; for it is exceedingly difficult to insert longer screws with slotted heads into hard wood without damaging the screw. No. 12 screws from 2 to 2½ inches long are the most economical sizes, so far as direct holding power alone is concerned.

A small crack or season check in the timber near a screw does not appreciably reduce the holding power, but a knot greatly increases it. There is practically no loss in holding power, due to removing a screw and reinserting it. In yellow

pine the holding power of screws in the sap wood is less than in heart wood, the difference being 30 percent or more. Since the elastic limit of timber is about 75 percent of its ultimate strength, a relatively low ratio between the ultimate and safe holding power for screws may be adopted provided the number of screws used in the connection is not too small.

An approximate value of the holding power may be conveniently computed by means of the following table which gives the ultimate holding power per linear inch of the total length of screw, expressed in pounds, upon the assumption that the entire threaded portion is engaged. For example, the holding power of a No. 12 screw, 2 inches long in white pine, is $2 \times 395 = 790$ pounds, and of a No. 20 screw, 3 inches long in yellow pine, is $3 \times 618 = 1854$ pounds. In order to determine the safe strength, these values must be divided by a suitable factor.

ULTIMATE HOLDING POWER OF WOOD SCREWS PER INCH OF TOTAL LENGTH WHEN INSERTED PERPENDICULAR TO THE FIBER.

Gage number	4	8	12	16	20	24	28	30
White pine	200	274	395	427	481	557	591	652
Yellow pine	295	400	505	547	618	675	732	
White oak	315	410	533	599	686	777	780	

The holding power of a drive screw equals about two-thirds of that of the common screw. It fails by overcoming the frictional resistance to turning. Lubrication greatly lessens its holding power, but is unnecessary, as it may be driven with ease into the hardest wood. With reference to its holding power it is intermediate between the nail and the ordinary screw.

Abstracts of the results of the tests referred to in this article were published in Transactions Association of Civil Engineers of Cornell University, 1898, vol. 6, page 63; and 1900, vol. 8, page 113, the discussion in the latter article relating to the data obtained by both investigators.

Another series of investigations was made at the Watertown Arsenal, including a little over 200 tests of wood screws, of Nos. 12 to 18, and ranging in length from $1\frac{1}{2}$ to $3\frac{1}{2}$ inches. The average resistance in pounds per square inch was found to be 922 in white pine, 1493 in yellow pine, 2598 in white oak, and 2338 in California laurel. Holes were bored equal to or slightly smaller in diameter than that at the root of the thread in the hard woods, but none were bored in the white pine. The average resistances in white and yellow pine with the screws inserted parallel to the fibers and no holes bored are 71 and 81 percent, respectively, of the corresponding values given in the preceding sentence. Since the same pieces of white and yellow pine were used as for the tests of nails, the corresponding resistances may be compared directly, showing that wood screws are about 2.3 times as effective in direct holding power as cut nails. For the detailed record of tests see Tests of Metals, etc., 1884, page 448.

Prob. 12. A No. 24 wood screw, having a total length of 4 inches, has an effective threaded length of 2.48 inches. The outer and inner diameters of the threaded part are 0.368 and 0.253 inch, respectively, and there are 7 threads per linear inch. Compute the resistance to withdrawal for an allowable compression of 390 pounds per square inch on the sides of the fibers of the wood into which it is inserted, on the assumption that the compressive area for a thread equals its horizontal projection.

ART. 13. HOLDING POWER OF LAG SCREWS.

The holding power of lag screws was investigated by W. M. MACPHAIL and T. T. IRVING, the results being published in Transactions Canadian Society of Civil Engineers, 1899, vol. 13, page 141. The screws were $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, and $\frac{3}{4}$ inch in diameter, while Canadian red pine (Norway pine) and white oak were used as representatives of the soft and hard woods, respectively, the total number of tests being 317.

Two sizes of holes were bored for each diameter of screw, one

equal to the diameter of the screw at the root of the thread and the other one-sixteenth inch larger. In the pine the holding power for the larger holes averages 95 percent of that for the smaller holes, while in the oak the corresponding relation is 105 percent. It appears that the insertion of the threads in the hard wood causes more injury to the fibers in the case of the smaller hole, and thus reduces the resistance of the wood when the screw is pulled out.

By splitting the block and examining the wood around the screw it is found that when the hole is bored too small, the fibers are crushed, but when it is of the right size, the thread in the wood looks clean cut, the fibers are pressed, and the fit of the wood on the screw resembles the appearance of a nut on a bolt.

The holding power was also found to vary directly as the diameter of the screw, which is the same relation as that given in Art. 12 for smaller sizes of wood screws. When the screws are inserted parallel to the fibers, the holding power is about 108 percent of that when inserted perpendicular to the fiber in the case of pine, and about 95 percent in the case of oak.

The holding power in the pine was found to vary directly as the depth of penetration of the threaded portion of the screw. In no case was the screw driven deeper than the threaded length. In the case of the oak, however, this relation is not so close. For screws driven $4\frac{1}{2}$ inches the holding power per linear inch is about 8 percent less than when driven only 3 inches. Since the screws were stressed nearly or quite to their ultimate strength, as several of them actually ruptured before the wood yielded, the unequal elongation of the screws apparently caused an unequal distribution of the pressure on the threads. For safe loads the elongation is much less, and the distribution may be regarded as uniform for practical purposes. The average ultimate holding power in the red pine was found to be about 780 and in the white oak about 1140 pounds per square inch.

In the report entitled Tests of Metals, etc., at the Watertown Arsenal for 1902, page 563, are given the results of 252 tests of the holding power of lag screws, four of them in yellow pine, while the rest are about equally divided between Douglas fir and white oak. The diameters of the screws range from $\frac{1}{2}$ to 1 inch in diameter. Three sizes of holes were bored for each diameter of screw, one equal to the diameter of the screw at the root of the thread, and the others respectively $\frac{1}{16}$ and $\frac{1}{8}$ inch smaller. In white oak there was practically no increased resistance due to the smaller holes, but in Douglas fir the average increase in resistance for the smallest holes was 5.2 percent over that for the largest, which equals the diameter at the root of the thread. Therefore, considering the greater effort required to insert the screw into the smaller hole, it is best to have the hole the same diameter as the core of the screw. It was also found that there is practically no difference between the average resistance for the common form of screw thread and the ratchet thread.

In this point the resisting area is computed as the area of a cylinder with a diameter equal to that of the outside of the thread as actually measured, instead of the nominal diameter, and a length equal to that of the undiminished threaded portion of the screw, thus excluding the tapering end. The average ultimate resistance in pounds per square inch is given in the following table:

RESISTANCE OF LAG SCREWS.

(Expressed in pounds per square inch)

Diameter of screw	1/2	5/8	3/4	7/8	1 inch
Diameter of hole	3/8	1/2	9/16	5/8	13/16 inch
In Douglas fir	869	657	689	882	593
In white oak	1603	1293	1232	1137	1175

The average resistance for all sizes of lag screws in Douglas fir is 739 and in white oak 1287 pounds per square inch. The corresponding value for the four tests in yellow pine is 777 pounds per square inch.

The average ultimate resistance per linear inch of penetration of thread is as follows:

RESISTANCE OF LAG SCREWS.

(Expressed in pounds per linear inch.)

Diameter of screw	1/2	5/8	3/4	7/8	1 inch
Diameter of hole	3/8	1/2	9/16	5/8	13/16 inch
In Douglas fir	1268	1348	1561	2291	1836
In white oak	2377	2484	2919	3185	3612

The corresponding resistance for the four tests of $\frac{3}{4}$-inch lag screws in yellow pine is 1830 pounds.

Since the average holding power in white oak expressed in pounds per square inch is 74 percent greater than in Douglas fir, it appears as if the resistance should be considered as proportional to the corresponding safe unit stress in compression on the side of the fiber. This relation is sufficiently precise to find the corresponding holding power for lag screws in other kinds of wood.

In the Watertown tests the resistance was also observed after the lag screws were withdrawn $\frac{1}{8}$, $\frac{1}{4}$, and $\frac{1}{2}$ inch, respectively, while for a part of the tests the corresponding distances were $\frac{1}{8}$, $\frac{3}{8}$, and $\frac{7}{8}$ inch. The resistances during withdrawal are given in the following table expressed as percentages of the ultimate holding power:

Distance withdrawn	1/8	1/4	3/8	1/2	7/8 inch
Resistance	92	76	55	44	14 percent

These results show that even in cases of failure the reduction in the holding power of lag screws is gradual.

A small number of tests was previously made at the Arsenal, the results of which may be found in the Tests of Metals, etc., 1894, page 448. The average resistance for 26 tests in white pine is 781, for 7 tests in yellow pine is 966, and for 4 tests in white oak is 1232 pounds per square inch. In white pine the sizes of screws tested are $\frac{5}{8}$, $\frac{1}{2}$, and 1 inch in diameter, while in

the other two kinds of wood only the 1-inch size was used. The record shows half-tone illustrations of the effect on the fibers of white pine and white oak, respectively, of pulling out 1-inch lag screws with the testing machine.

In the same publication for 1887, page 934, are given four additional tests, with diameters of $\frac{1}{2}$ and $\frac{5}{8}$ inch. The resistance in pounds per square inch is 991 in yellow pine, 613 in white cedar, and 735 in hemlock, the last value being an average of two tests.

A few tests made by P. LOBBEN were published in the American Machinist, vol. 18, page 964, Dec. 5, 1895. The sizes range from $\frac{7}{8}$ to $\frac{1}{4}$ inch in diameter; 7 tests in spruce give an average ultimate resistance of 735 pounds per square inch, and 1 test each in chestnut and pitch pine give 691 and 783 pounds per square inch, respectively.

Some experiments made at Sand Beach, Mich., give the holding power of a 1-inch screw in white pine, with a $\frac{7}{8}$-inch hole bored, as 435 pounds per square inch. This value is the average for 12 tests in two sticks of timber. When the holes are bored either $\frac{13}{16}$ or $\frac{15}{16}$ inch, the resistance is smaller. See Report of Chief of Engineers, U.S.A., 1884, page 2064.

Prob. 13. Refer to Tests of Metals for 1902, and compute the average percentage of increased holding power of each size of lag screw in Douglas fir due to boring the holes smaller than that of the screw at the root of the thread.

ART. 14. DOWELS.

A dowel is an iron or wooden pin extending into, but not through, two members of a structure to connect them. The dowel is usually placed perpendicular to the surface of contact. It is most frequently employed to connect the end of one timber to the side of another, to prevent the lateral displacement of the former. The dowel differs from a drift bolt in general by being larger in diameter and shorter in length. It has

neither point nor head, and is used in the same form as cut from the rods, provided it is straight.

In Fig. 14*a* the dowel holds the tip of the follower pile in place on the butt of the lower pile. In Fig. 14*b* two dowels are used in the same dowel joint, and Fig. 14*c* shows the hip joint of a Howe bridge truss, which contains two dowels and

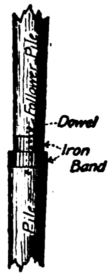


FIG. 14*a*.

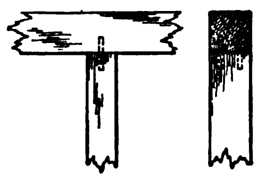


FIG. 14*b*. Dowel Joint.

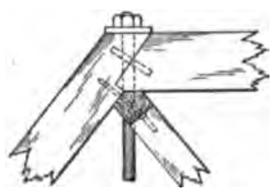


FIG. 14*c*.

one boat spike. The upper dowel is 1 inch in diameter and 8 inches long (Eng. News, vol. 23, page 402, April 26, 1890). See also the dowel joints in the Howe truss of an exposition building illustrated in Fig. 76*a*.

In its report in 1892 on framing timber trestle bridges, a Committee of the American Association of Railway Superintendents of Bridges and Buildings made the following reference to dowels: "The second method of holding together trestle joints is by means of dowels. . . . This method, although cheap as to first cost, has many objectionable features. First, it does not give a rigid joint, and in renewals it is seriously in the way of renewing defective timber where the time between trains is limited, for where a tenon can be readily cut off with a saw, the dowel would have to be cut off with a heavy chisel bar and a maul, or the wood cut away from it sufficiently to release the timber."

In trestles the sizes of dowels usually range from 1 to 2 inches in diameter and from 6 to 12 inches in length, but occasionally

these limits are passed. Holes are bored slightly less in diameter than the dowels, so that they will fit closely after the timbers are firmly driven together. The dowels and the whole joint surface should be thoroughly painted or covered with some preparation of coal tar, carbolineum, or other preservative before they are put together, thus making the joint practically water-tight and preventing the early decay of the timber. In this respect the dowel joint is superior to the mortise and tenon joint, since with the class of timber used in trestles it is impracticable to make the latter joint tight for any length of time.

The dowel, however, does not make as rigid a joint as the mortise and tenon, and some difficulty is experienced in erecting large trestle bents framed in this manner. After the bent is in place, however, and the diagonal and longitudinal braces attached, there is practically no deficiency in rigidity. The time and expense for framing with dowels is much less than with the mortise and tenon, while a much larger surface of contact is available for bearing in timbers of equal size.

It is customary to extend the dowel equal distances into the two timbers joined, but good design requires a longer penetration into the end of one timber than into the side of the other, since there is a greater tendency to split the former.

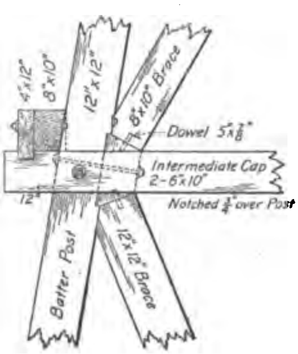


FIG. 14*d*.

Additional examples of the use of dowels in framing may be seen in Plate III, in the connection of the floor beams to the foundation posts, and in a roof truss in Art. 69. Fig. 14*d* shows an intermediate joint of a two-story trestle in which the braces are compression members. See also Engineering News, vol. 37, page 331 (inset), May 27, 1897, where the plan of the upper

coping course shows the use of $1\frac{1}{4}$-inch round steel dowels, 12 inches long, for the face stones of the Brooklyn pier of the Williamsburg bridge.

Dowels are also employed in trestle construction to anchor the cross-tie to the stringers when the stringers are covered with galvanized iron in order to prolong their life. The size used for this purpose is $\frac{5}{8}$ inch in diameter and 5 inches long, the holes being bored $\frac{11}{16}$ inch in diameter to insure a driving fit. See an article on Protection of Members in Wooden Bridge Structures, Engineering Record, vol. 59, page 637, May 15, 1909. A dowel screw is shown in Fig. 11*f*.

Prob. 14. Prepare a list of the illustrations in Chaps. IV and V in which dowels are shown.

ART. 15. WOODEN PINS AND TREENAILS.

Wooden pins are round pieces of hard wood inserted through the timbers of a joint to hold them together. A common example of its use is that in which the pin prevents a tenon from drawing out of a mortise (see Art. 30).

Treenails are slender pieces of hard wood used in a similar manner to iron nails to fasten boards or timbers together. They are used where iron nails or spikes would cause injury by rusting, and where copper nails are too expensive. Treenails are slightly tapered (Fig. 30*d*) to facilitate driving and are made of different diameters and lengths according to the sizes of the timbers which they unite. Sometimes treenails are used as pins in bringing the surfaces of joints firmly to their bearings and retaining the timbers in place.

An illustration of the use of treenails is given in Fig. 15*a*, which shows the detail of a portion of the wooden curbing for the concrete pivot pier foundation of the Charlestown bridge at Boston. The curbing has a mean diameter of 75 feet and is 32 feet high. Each course consists of 24 spruce planks 3 by 12

inches, each plank being about 10 feet long. The planks were planed on one side to an even thickness, sawed to proper length and level, and spiked and treenailed together. The treenails are

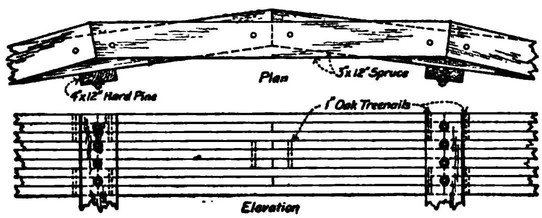


FIG. 15*a*. Wooden Curbing for Pier Foundation.

made of oak and one inch in diameter. See Engineering Record, vol. 38, page 186, July 30, 1898.

Some small-size shearing tests across the grain on specimens $\frac{5}{8}$ inch in diameter were made by JOHN C. TRAUTWINE, the results of which were originally published in Journal of the Franklin Institute, Third Series, vol. 79, page 105, February, 1880. Two tests in double shear were made for pins of 24 different kinds of wood. Some of the largest values expressed in pounds per square inch are: ash, 6280; beech, 5223; birch, 5595; dogwood, 6510; gum, 5890; hickory, 6045 to 7285; locust, 7176; maple, 6355; white oak, 4425; live oak, 8480; and Southern yellow pine, 5735.

In a table given in Manual of Railroad Engineers, by GEORGE L. VOSE, it is shown that the resistance of seasoned oak treenails to shearing across the fiber is greater when they connect 6-inch planks than 3-inch planks. Apparently the treenails are not allowed to bend as much in the thicker planks before they rupture. For diameters of 1, $1\frac{1}{4}$, $1\frac{1}{2}$, and $1\frac{3}{4}$ inches the ultimate shearing stresses in pounds per square inch were found to be 4282, 4130, 3435, and 3203 when in the 3-inch planks, and 4538, 4130, 3910, and 3967 when in the 6-inch planks.

The species most frequently used are oak, locust, and hickory.

In designing wooden pins and treenails it is necessary to consider the bearing on the side of the fiber for the pin as well as the bearing either on the side or end of the fibers for the wood in which the hole is bored, depending upon the direction in which the pressure is transmitted. In construction it should be remembered that the resistance to bearing on the side of the pin is greater when the line of pressure is perpendicular to its annual rings.

For an example, in the investigation of the strength of a pin, let oak pins 2 inches in diameter connect two planks each $2\frac{7}{8}$ inches thick which tend to slide past each other longitudinally. The distribution of the pressure or loading on the pin in each

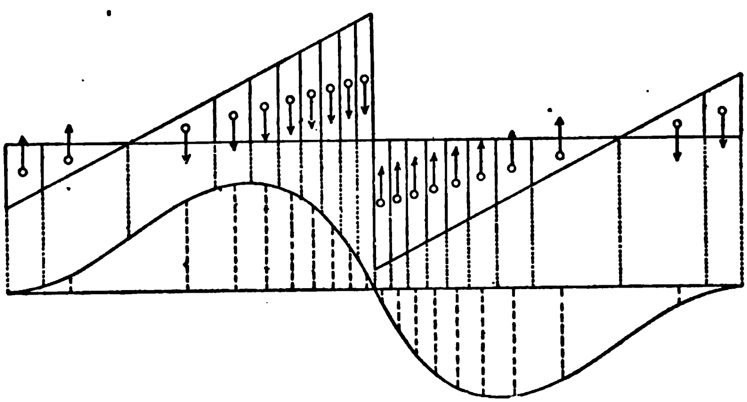


FIG. 15*b*. Bending Moment Diagram for a Wooden Pin.

plank is like that of a railroad spike as illustrated in Fig. 8*b*, provided the distance e equals zero. P therefore indicates the magnitude of the vertical shear in one pin in the section between the two planks. Formulas (1) in Art. 8 give values of $c = \frac{1}{8}l$ and $a = \frac{3}{8}l$ when $e = 0$, the distance l being the length of the pin in one plank. The loading of the pin in both planks is indicated in Fig. 15*b*, and the same diagram also gives the bending moment diagram, which was constructed graphically. The load areas were so subdivided as to make each one represent a

load of $\frac{1}{8}P$. The pole distance was taken equal to $\frac{1}{2}P$, and the maximum moment ordinate was found to measure 0.85 inch. The maximum moment is therefore $0.85\ P/2 = 0.425\ P$ pound-inches, P being expressed in pounds.

The section modulus of a 2-inch pin is 0.7854 inches^3, and for a unit stress in the outer fiber of 1500 pounds per square inch, the resisting moment is 1178 pound-inches. Equating the resisting moment to the bending moment, the value of the shear P is found to be 2770 pounds. For a unit stress in shear across the fibers of the pin of 1000 pounds per square inch the value of P is 3142 pounds.

Assuming that the bearing of a round pin of oak is equivalent to 0.74 times the bearing on a square pin, and that the allowable compression on the side of the fiber is 700 pounds per square inch, the maximum bearing on the pin per linear inch is $0.74 \times 2 \times 700 = 1036$ pounds. The load $4P/3$ is distributed over a distance of 1.92 inches, but the maximum load is twice the average load, hence $4P/3 = (1036/2)\ 1.92$, whence $P = 995$ pounds. This computation shows that the compression on the side of the fibers of the pin really determines the magnitude of the shear between the planks which it can resist. The use of hollow metal pins, cut from pipe, would materially increase the strength of the joint.

Prob. 15. If a pin made of extra hydraulic tubing with an inside diameter of 1.49 inches and a thickness of 0.20 inch be substituted for the oak pin in the example given in this article, compute the shear between the plank which it will resist.

ART. 16. WOODEN KEYS AND WEDGES.

Wooden keys are inserted in some types of joints to bring the surfaces into close contact or to prevent some piece from being withdrawn. Illustrations of their use are given in Figs. 27*e*, *k*, *m*, *n*, 30*k*, and 31*k*.

When a key is used to prevent two adjacent timbers from sliding longitudinally with respect to each other, it is best to

design the key with the fibers parallel to those of the timbers in which it is inserted, as indicated in Fig. 16*a*. The longitudinal pressure of the timbers is taken directly by the ends of its fibers, while the key is subject to shear parallel to its fibers, and to rotation which is resisted by bearing on its sides. The compression on the half of each upper or lower side of the key is

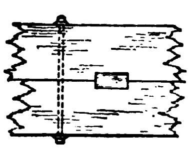
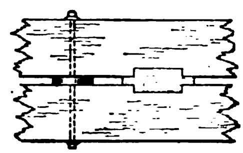



FIG. 16*a*. Wooden Key. FIG. 16*b*. Key with Shoulders.

distributed as the ordinates to a straight line, the maximum pressure being at the end. The tendency of the key to rotate will cause the timbers to separate unless they are held together by one or more bolts. The tension in the bolt is therefore directly related to the pressure on the sides of the key.

For example, let it be required to design a rectangular wooden key of western hemlock to resist a longitudinal shear of 10 000 pounds between two timbers 6 inches wide. The specified unit stresses, expressed in pounds per square inch, are: Shear parallel to the fiber, 240; compression on the ends of the fiber, 1800; and compression on the side of the fiber, 330. Placing the fibers of the key parallel to the timbers, the length required to resist shearing is $10\,000/(6 \times 240) = 6.945$ or 7 inches. The required depth of the key is $2 \times 10\,000/(6 \times 1800) = 1.852$ or $1\frac{7}{8}$ inches, which is preferably increased to 2 inches, so as to cut the keys from 2-inch plank.

The moment of rotation due to the eccentric compression (Fig. 16*c*) at the ends of the key is $10\,000 \times 2/2 = 10\,000$ pound-inches. To find the length required to keep the allowable pressure on the sides of the fibers within the allowable limit of 330 pounds per square inch, let the length be designated

by l. The total compression on each side of the key is $(330/2)(6l/2) = 495\,l$ pounds, and the lever arm of the resultant couple is $2\,l/3$, making the moment $330\,l^2$. Equating the two moments, there is obtained $l = 5.505$ or $5\frac{5}{8}$ inches. As this value is less than that required for shear the latter will govern, making the length to be adopted 7 inches.

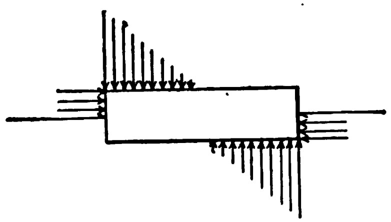


FIG. 16c. Compression on a Key.

Wedges are extensively used under falsework upon which arches and other bridges are erected so that when the structure is ready to become self-sustaining, the support of the falsework may be gradually released by slowly withdrawing the wedges.

Large wedges are usually made of oak wood because of its large resistance to compression on the side of the fiber. For the same reason small wedges are frequently made of hickory, this wood being also very tough. For large wedges under heavy loads a taper of 1 in 10 may ordinarily be used or in extreme cases 1 in 20. Under light loads the taper is often increased to about 1 in 6, or 2 inches per foot. In order to compute the static force necessary to drive up or to release wedges, it is necessary to know the friction between surfaces of wood under pressure.

The friction of wood upon wood at high pressure was determined experimentally by E. H. MESSITER and R. C. HANSON in 1894, and an abstract of the results was published in Engineering News, vol. 33, page 322, May 16, 1895. Eighty-one tests were made at pressures ranging from 100 to 1000 pounds per square inch, chiefly on yellow pine. When the sides of the fibers are placed longitudinally on each side of the rubbing surface, the coefficient of friction is 0.365 when the surfaces are rough as they come from the lumber yard, and 0.215 when the

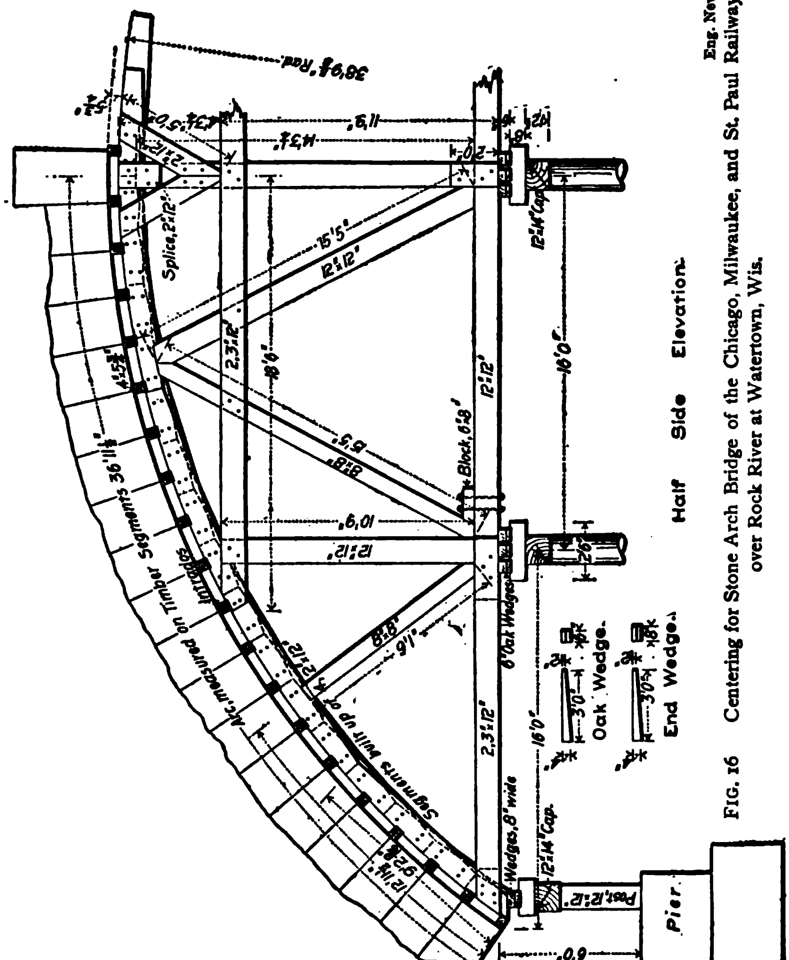


FIG. 16 Centering for Stone Arch Bridge of the Chicago, Milwaukee, and St. Paul Railway over Rock River at Watertown, Wis.

FIG. 16*d* gives the dimensions and positions of the wedges under the centering for a stone arch bridge. The wedges were greased and secured in place to prevent slipping. For a half-tone view of the centering and arch ring under construction see Art. 75.

timbers are planed smooth. For smooth or sandpapered surfaces, the coefficient of friction is 0.227 when the ends of fibers rub against the sides of fibers, and 0.323 when the ends of fibers rub against the ends of fibers. When the fibers are inclined on each side of the rubbing surface, either in the same or in opposite directions, the coefficient ranges from 0.255 to 0.315, the larger value being for an inclination of 30 degrees, and the smaller one for 45 degrees. The mean value for 64 tests on yellow pine was found to be 0.290. In every case the surfaces were found to be polished after the completion of the test; and in the case of very pitchy specimens the specimens were found to be sticky, especially after tests at very high pressures. A small number of tests was also made on fairly dry spruce which gives a mean value of 0.422 as the coefficient of friction.

In striking the centers of the Piney Branch arch bridge at Washington, D.C., the coefficient of friction was observed by W. J. DOUGLAS, its value being about 0.4. The wedges had been greased, but it was thought that the grease did not facilitate the striking. The maximum pressure on the large wedges at the center of the span was 280 pounds per square inch, while that on the other wedges varied from 160 to 200 pounds per square inch. See Art. 75 and Engineering News, vol. 57, page 682, June 20, 1907.

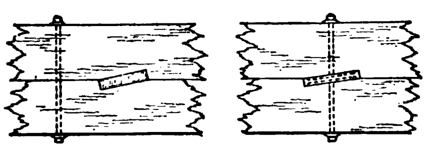


FIG. 16*e*. Wooden Key. FIG. 16*f*. Cast-iron Key.

By inclining the keys as in Fig. 16*e* the shear parallel to the fibers is avoided, and the compression at each end is distributed over its full depth. Fig. 16*g* shows the use of keys placed in this position in the construction of double-leaf gates for the Lockport locks of the Chicago Drainage Canal. Each leaf is built of single horizontal courses of Douglas fir fastened together by white oak keys and vertical through bolts. The weight of

the leaf tends to sag its movable end and to cause the courses to slide on each other. This is resisted by the keys and by the

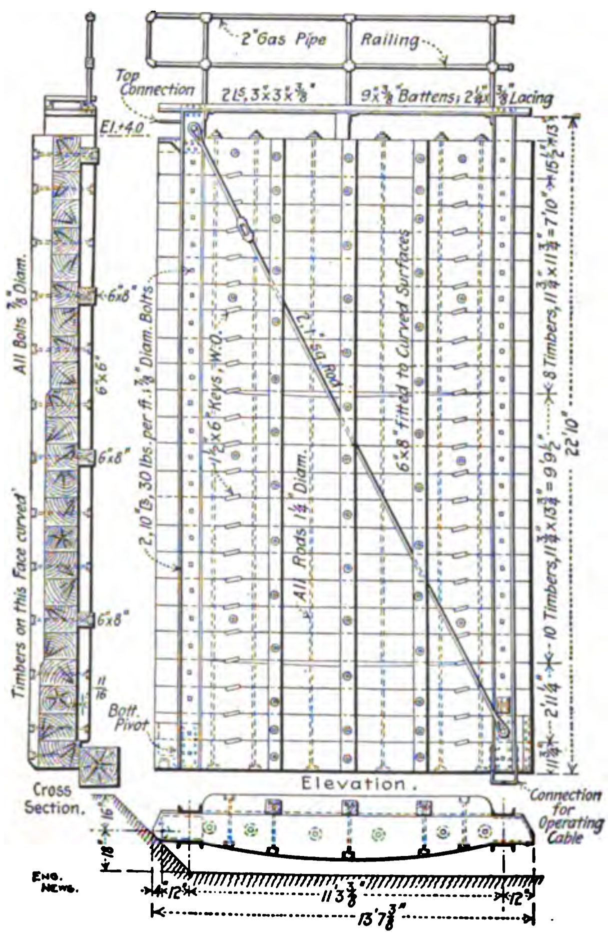


FIG. 16g. Lock Gate on Chicago Drainage Canal.

diagonal rods which are drawn tight by means of turnbuckles. See Engineering News, vol. 60, page 512, Nov. 12, 1908.

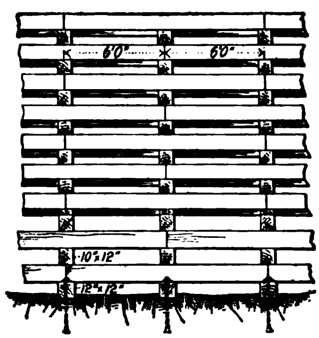
Round keys of white oak 2 inches in diameter were extensively

used in the temporary wooden building construction at the St. Louis Exposition. They were employed instead of rectangular keys in fish-plate splices in columns and truss chords and between the main timbers forming composite columns (see Engineering News, vol. 51, page 478, May 19, 1904). For an illustration of their use in similar construction at the Pan-American Exposition see Figs. 50*c* and 74*f*.

Prob. 16. Compute the total force required to release the large wedges of the Piney Branch arch centers.

ART. 17. ANCHOR BOLTS.

The object of an anchor bolt is to fasten in place or hold down to the supporting masonry some part of a structure, or machine. An anchor bolt has a thread cut at its upper end to engage a nut while the lower end is embedded in the masonry. The bolt is embedded by drilling a hole in the masonry, setting the bolt in position, and filling the hole with melted lead or sulphur or a grout of neat cement. Since cement has come into such extensive use in construction, it has largely replaced other material for this especial purpose. Sometimes the anchor bolts are built into position as the stone or brick wall is constructed, in which case the bolt has a nut at the lower end also, which acts against a bearing plate.



Side Elevation.
FIG. 17*a*. Anchorage of Wooden Dam.

In some experiments made by E. F. MINER to determine the relative value of Babbitt metal, lead, and sulphur, in fastenings which were not to penetrate over 6 inches into stone, it was found that sulphur gave the most satisfactory results. In two tests with sulphur setting, the stone broke under a stress of 950

pounds per square inch of embedded surface of bolt. See Engineering Record, vol. 26, page 47, June 18, 1892.

In Engineering News, vol. 24, page 53, July 19, 1890, are given the results of 14 tests, divided equally between bolts ⅞ and 1 inch in diameter, set in holes 1⅜ and 1⅝ inches in diameter drilled 3½ feet deep into a ledge of solid limestone. The ends of the rods were ragged. The cement setting was shown to be superior to that of lead and sulphur, both as to strength and ease of application. Four additional tests were then made with 1 and 2-inch bolts, set in stones 2 inches thick, two of the bolts being threaded and two plain. When the bolts were pulled very slowly, the first indication of yielding occurred at about 650 and 520 pounds per square inch of embedded surface for the 1 and 2-inch bolts respectively. On continuing the tests at greater speed, the stones finally split at considerably higher stresses. The threaded bolts indicated no superiority over the plain ones.

A few tests made by ROBERT MOORE are reported in Engineering Record, vol. 23, page 209, Feb. 28, 1891. In one case a 2-inch plain rod set 11½ inches deep with Portland cement began to yield at a resistance of 470 pounds per square inch, but the cement setting did not entirely part at 927 pounds per square inch, when the stone broke. In another case with a threaded bolt the stone began to yield and finally broke when the resistances were 443 and 692 pounds per square inch respectively, without developing the full strength of the cement joint.

A larger number of experiments was made by E. S. WHEELER at St. Mary's Canal, and are recorded in Report of the Chief of Engineers, U.S.A., 1895, pages 2917 and 2940. The rods were plain, being ½, 1, and 1¼ inches in diameter, both round and square. They were embedded about 8 to 10 inches in mortar composed of one part of Portland cement and two parts of limestone screenings passing ⅜-inch slits. The mean adhesive resistance for 3 tests of each size and form of rod was found to

vary from 434 to 562 pounds per square inch, the general mean for 6 sets being 511 pounds per square inch. One-inch square twisted rods with 1 to 3 turns in 8 inches, developed less than 8 percent greater resistance than the plain rods. Some additional tests were made by substituting ordinary river sand for the limestone screenings, giving an average adhesive resistance of 264 pounds per square inch. On using four parts of sand instead of two, the adhesion was reduced more than half.

A series of tests was made in 1905 by EDWARD HOLMES in the civil engineering laboratory of Cornell University. The bolts were of $\frac{7}{8}$-inch wrought iron, 18 inches long, with split bushing at the lower end. They were set 6 inches deep into 10-inch cubes of shale rock known locally among stone cutters as Ithaca marble. The holes were cut 2 inches in diameter. Cement grout, sulphur, and lead were used respectively in setting 5 bolts each. The cement was allowed to set 14 days before testing, except one which was tested after 8 days. The bolts set in sulphur or lead were tested after 24 hours.

The average results of the tests are as follows: In cement the bolts showed signs of loosening at 16 340 pounds, and pulled out at 21 640 pounds; for the setting in sulphur the corresponding values are 13 140 and 18 480 pounds; and in lead, 8000 and 10 980 pounds. The minimum resistances at first signs of loosening are 12 000 pounds for the cement tested after 8 days only, 7600 for the sulphur and 7500 for the lead. Based on the adhesion of the cementing material to the stone, the resistance at the yield point is 408 pounds for the cement, 328 for the sulphur, and 200 for the lead. In one case the bolt broke, the setting being in cement, and the ultimate resistance 30 000 pounds, although the first sign of yielding occurred at a load of 19 500 pounds. The superiority of a cement setting is clearly demonstrated by these tests.

The results of other experiments on the adhesion of rods or

bars in cement or concrete are given in various treatises on concrete construction.

Various patented forms of anchor bolts, some of which are of the expansion type, are advertised in architectural and engineering periodicals. One of these has the novel feature of being hollow, and was first used on the regulating works of the Chicago Drainage Canal. In an illustrated description in the Railroad Gazette, vol. 32, page 606, Sept. 14, 1900, it is stated that holes are drilled in the masonry the same as for ordinary anchor bolts, and then the hollow bolts are attached to the ironwork, which is lined up to final position. The edges of the ironwork joining the masonry surface being pointed with stiff mortar, a thick grout of neat Portland cement is forced through the bolt by a hand pump. In this way the ironwork can be set in the final position before the anchor bolts are cemented, while no play is required in the bolt holes of the ironwork. This latter feature is especially valuable where shearing forces are to be opposed. The safe holding power of these bolts may be computed for tension at 15 000 pounds per square inch of net section for the iron, and at 200 to 250 pounds per square inch of embedded bolt surface.

Another type consists of a split bushing with a thread on the inside and projecting points on the outside to be used in securing a screw bolt to stone or brick masonry. An illustrated description may be found in Engineering News, vol. 39, page 286, May 5, 1898. A modification of the same type, having serrated sleeves with a lag screw inserted between, is especially intended to anchor into timber. See Engineering News, vol. 49, page 457, May 21, 1903.

Prob. 17. To what depth is a $\frac{3}{4}$-inch anchor bolt to be embedded in a concrete wall, in order to develop its working stress of 15 000 pounds per square inch. The al.owable value of the adhesion to be taken is one-fourth of the ultimate value.

ART. 18. METAL STRAPS AND PLATES.

In many kinds of joints some fastening is necessary to hold the timbers together. For this purpose bolts are most frequently employed, while sometimes spikes, lag screws, or other metal fastenings are used. Metal straps are used in some cases where the simpler fastenings are not conveniently applied, or where the tendency to displacement may require a stronger connection. Fig. 18*b* shows a strap holding the housed end of

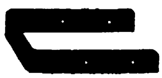


FIG. 18*a*.

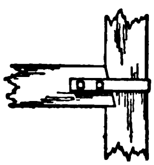


FIG. 18*b*.

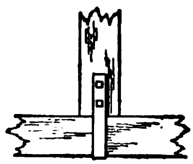


FIG. 18*c*.

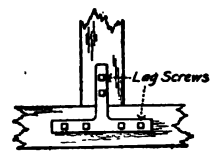


FIG. 18*d*.

a horizontal brace to the post, the strap being bolted to the brace. In Fig. 18*c* the strap holds the lower chord of a roof truss from sagging. An alternative arrangement is shown in Fig. 18*d*, where a strap of a T-form is attached to each side with lag screws. Sometimes more elaborate and artistic forms are used on exposed roof trusses. Straps are also employed on lock gates and on heavy doors subjected to shocks.

Metal straps are used to a limited extent in the framing of wooden bridge trestles. The report of a Committee of the Association of Railway Superintendents of Bridges and Buildings in 1892 includes the following paragraph: "The fifth plan is that of using bent metal straps, usually called pile straps, being straps about ½ by 2½ inches by 5 feet in length, bent to engage the trestle caps or sills with legs 2 feet in length on either side of the post, and secured with heavy spikes to the cap or sill and posts. The fastening, for pile trestles more especially, is being used by some of the best railroads with good results, and being easily removed when the timbers become defective, it is considered a good device. In using it to secure the caps of pile

trestles, the piles, after being cut off to receive the caps, should be squared or sized down to fit snugly the dapping or boxing in the lower side of the caps, which should be made at least one inch in depth. No dowels should be used except in case of a refractory pile which could not be held in place without them. This plan of securing the caps for pile trestle bents makes a secure joint, and admits of easy and speedy renewal of timbers when renewals become necessary."

In Fig. 18*e* straps are used in connection with stepping and housing on a track buffer. As indicated, the ends of two straps

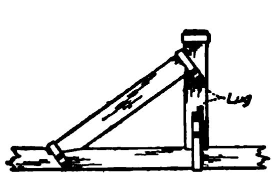


FIG. 18*e*. Track Buffer.

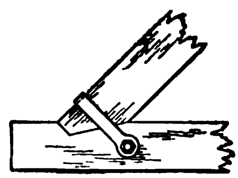


FIG. 18*f*. Heel Strap.

are turned inward to form lugs to engage slots cut in the timber, but it is questionable whether they add materially to the strength of the connection, since they are so close to the bolts. In Fig. 18*f* the bent strap has an eye forged on each end to engage a pin or bolt of relatively large diameter. This form is

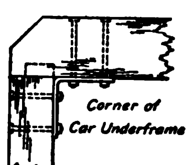


FIG. 18*g*.

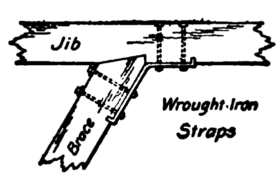


FIG. 18*h*. Joint in Jib Crane.

called a heel strap and is occasionally used on the end joints of roof trusses. Whether a heel strap may be expected to resist any part of the horizontal thrust of the upper chord will be discussed in Art. 33.

In Figs. 18*g* and *h* the straps or plates are bent around an angle formed by the timbers to which they are bolted. The

former joint is at the corner of a car underframe, while the latter is one of the joints of a foundry crane which has been in service for many years, but its joints appear to be as firm as when built. A similar application of bent straps to another kind of derrick is illustrated in Engineering Record, vol. 36, page 10, June 5, 1897.

Fig. 18*i* indicates how metal plates are sometimes used in connecting the timbers of a building frame. See Engineering News, vol. 28, page 447, Nov. 10, 1892. A number of straps and plates are shown on the plans of a derrick and tower in Engineering News, vol. 43, page 164 (inset), Mar. 8, 1900, and in Engineering Record, vol. 41, page 576, June 16, 1900.

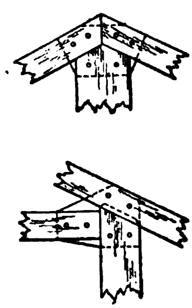


FIG. 18*i*. Metal Plates.

Since 1907 steel straps and plates have been adopted in the connections of heavy false work for bridge erection, having wooden posts and horizontal struts, combined with adjustable diagonal rods. In building up the towers all the stories are made of standard lengths, except the one at the top, and are so connected with bolts and pins that they can be taken down and shifted to other places.

The application of bent metal plates to wooden trestles is illustrated in Fig. 18*j*, the two left-hand diagrams showing the details more clearly in isometric projection. See Engineering News, vol. 18, page 326, Nov. 5, 1887. Their construction and advantages are described as follows in the report of the committee previously referred to in this article: "The fourth method, that of using bent metal plates, has given very good results. These plates are made of thin boiler-plate iron, generally one-fourth of an inch in thickness, cut and bent with flanges in opposite directions at right angles to opposite sides of the plates, one set of flanges engaging the cap or sill, and the other set

engaging the posts of the trestle bent. They can be rapidly and cheaply made to suit any sized timber and any angle at which the timbers meet, leaving but little work to do in the field; namely, that of squaring the timber and sizing it down to fit

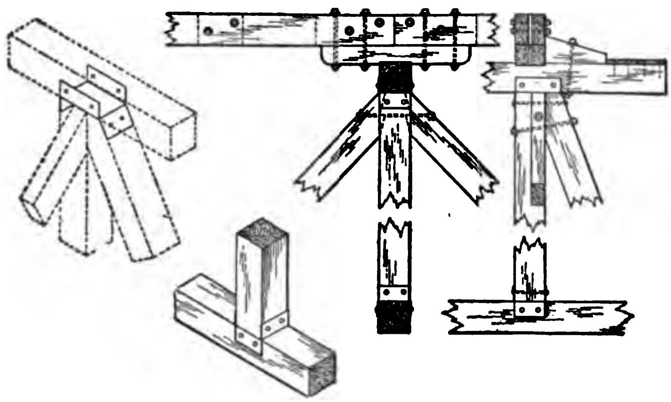


FIG. 18*j*. Wrought-Iron Plates in Trestle Construction.

between the flanges on the plates, and spiking them in place. This method produces a joint of considerable strength, and is free from many of the objectionable features found in the dowel and drift-bolt joints."

Strap bolts consist of a strap at one end and a bolt at the other as shown in Fig. 18*k*. The detail in Fig. 18*l* shows their application in fastening the back braces of a pile driver to the bed timbers. The ends of the straps are bent inward to form a lug.

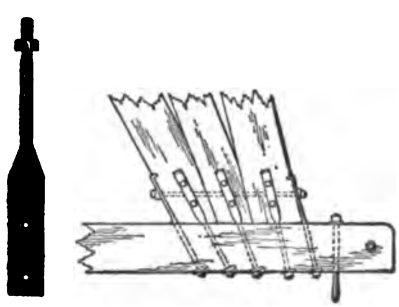


FIG. 18*k* and *l*. Strap Bolts.

A cramp, illustrated in Fig. 18*m*, is an iron with bent ends, serving to hold two pieces together. It may be used either to bind together the ends of the

two pieces, as in a butt joint, or the end of one piece to the side of another. A cramp is also used in masonry to bind adjacent

FIG. 18m. Cramps.

stones together where great strength is required, as in the masonry of the pivot pier of a swing bridge or in a lighthouse.

Prob. 18. Explain the method of designing a strap bolt and its connections to the end of a timber, to resist a given tensile stress.

CHAPTER II.

JOINTS USED IN FRAMING

ART. 19. TABLED FISH-PLATE JOINT.

Timbers are joined in the direction of their length by the operation called splicing. The joints so formed are divided into three classes, of lapping, fishing, and scarfing. When a timber is to transmit only tension, fishing is more effective than either lapping or scarfing.

The best form of fishing for this purpose is that containing tabled fish plates, illustrated in Fig. 19*a*. The plates are also called indented fish plates. The projecting tables of the fish

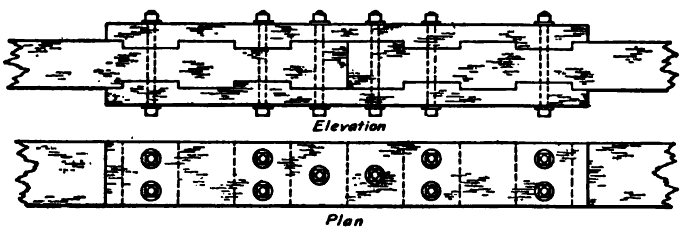


FIG. 19*a*. Tabled Fish-plate Joint.

plates engage corresponding notches in the main timbers. The bolts are required to keep the parts in position and to resist the tendency of the fish plates to bend.

If such a joint be tested to destruction it may fail: first, by the rupture of one or both of the main timbers; second, by the rupture of one or both of the fish plates; third, by the rupture of one or more of the bolts, which in turn would cause the failure of the fish plates by flexure; or fourth, by one or more washers

crushing the fibers of the fish plates and thus causing their failure by splitting or tearing. A main timber may fail: first, by tearing the fibers in the smallest cross-section which transmits the full stress, and this occurs at the bolts near the ends of the fish plates; second, by crushing the ends of the fibers on one side of each table; or third, by shearing the tables at their

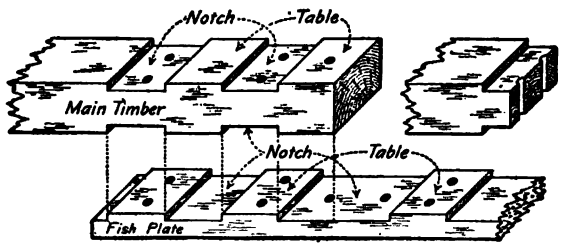


FIG. 19*b*. Parts separated to Show Notching.

bases, along the fibers. A fish plate may fail in the same manner. The failure of bolts of standard dimensions (see manufacturer's handbook or tables) occurs by rupture at the root of the thread.

The joint will be safe when all its parts are so designed that there is safety in every one of the respects named. Because the strength of a joint is equal to that of its weakest part, good design requires an equal degree of security in all of these respects. Since one part affects another, it is necessary to consider their mutual relations, together with the conditions involved in the actual construction of the joint.

THE NET TENSILE SECTION.

As the safe strength of the net section of the main timber must equal the given stress to be transmitted by the joint, economy requires that the gross section shall be no larger than that needed to furnish also the tables, the combined strength of which equals the given stress. If the notches be too deep, there is a waste of timber whose relative value depends upon the length of the timbers spliced. In practice the cost of two or

more designs may be compared and the most economical one adopted, the cost of both material and workmanship being taken into account.

It will be noticed from Fig. 19*b* that the plane containing the net section area of the main timber passes through two bolt holes, while that of each fish plate passes through only one. The two bolts next to the center of the joint are inserted to keep the parts in contact and to strengthen the joint while being handled and placed in its final position in the structure.

THE NUMBER OF TABLES.

The number of tables on each main timber as well as the number of those on each fish plate must evidently be an even number, and hence it is necessary to decide whether 2, 4, or 6 are to be adopted. The cost of workmanship increases almost directly as the number of tables. On the other hand the amount of timber decreases somewhat as the number of tables increases, since the height of the tables is thereby reduced. The ratio of the amount of material to the number of tables depends also on the length of the timbers to be spliced, and is therefore not a simple one. On account of imperfections in workmanship, the larger the number of tables the less likely they are to work together, or to receive equal stresses. If any table receives a larger stress than its share, the degree of security of the entire joint is thereby reduced.

Further, the slight inequality in the depths of the tables on the main timbers and of the corresponding ones on the fish plates, due also to defective workmanship, diminishes their effective bearing area, and this reduction is relatively greater in low tables than in high ones. Only in fine work can a carpenter be expected to work as close as a sixteenth of an inch, while in large framed joints the depths are usually expressed only to a full eighth of an inch. On the other hand, if the number of tables be reduced, the resultant line of stress is carried farther from the

axis of the main timbers. This increased deviation implies the development of larger bending moments to be resisted by the bolts, and their increased size reduces in turn the net tensile section of the timber.

Again, the length of the table is reduced by increasing their number. If the bearing is not uniform across the end of each table, the strength of the entire joint is thereby reduced; and if the joint were tested to destruction, it might fail by shearing off the tables by parts in succession without developing their full strength under proper conditions. To avoid this reduction it is advisable to limit the ratio of the length of a table to its width, and this ratio ought not probably to be less than two-thirds. On the other hand, the length should not be greater than about the width of the timber, in order that the bolts may readily keep the parts in contact.

THE BOLTS.

The bolts do not simply keep the parts in position, but they receive tensile stresses that depend directly upon the magnitude of the stress carried by the joint. Since the resultant of the tension in the net section of the fish plate does not coincide with that of the compression against the adjacent table, a moment is developed which tends to bend the fish plates. This moment is resisted by the bolts at each table, the center of rotation being in the surface of contact between the table of the fish plate and that of the main timber. It will be observed that in this surface the ends of the fibers engage one another.

The farther the bolt is from the center of rotation the less will be its tensile stress, and hence the smaller its diameter. In order that the joint shall not open between the fish plates and main timber when the joint is under its maximum stress the bolts must be drawn up to their full tension when the joint is constructed. If the bolt be not at the mid-length of the table, the fibers on its surface will be compressed unequally and thus

introduce initial bending in the fish plates. It is preferable, therefore, to place the bolts at, or very near to, the mid-length of the table, even though it requires slightly larger bolts than if placed near the farther end. If by placing a bolt just a little beyond the middle, its full safe strength can be utilized, it is sometimes preferable to using a larger bolt exactly at the middle which may have considerable excess of strength, while the latter may at the same time decrease the net section just enough to require an addition of a full inch to the gross depth of the timber.

Since the stress in a bolt is transmitted to a fish plate by the bearing of a washer, it is evident that there must be a limit to the width of fish plate which can be held effectively without warping or without transmitting the stresses too indirectly. If the timber be not wider than six inches, one row of bolts will answer. Two rows are used for a greater width, and three rows when the width exceeds fourteen inches. When two rows of bolts are used, it should be assumed that each row acts on its half of the width of the timber, and should be located on the center line of that half; and similarly for three rows.

To insure this condition the holes should be bored from $\frac{1}{16}$ to $\frac{1}{8}$ inch larger than the diameter of the bolts. The bearing area of the washer is to be computed. It should also be observed that the diameter of the hole in the washer is larger than that of the bolt (see Art. 2). The thickness of the washer may then be selected from the tables of standard or special sizes manufactured for the trade.

CONTROLLING CONDITIONS.

Whenever the structure imposes any controlling conditions on the proportions of the joint, some of the preceding considerations may have to be waived. For instance, if the joint must be shorter than the length previously determined, it can only be made so by increasing the width, since the shearing area

is fixed. The slight extensions of the fish plates beyond their end tables are made to prevent the outer fibers of the main timbers from splitting off readily.

APPLICATIONS.

Figure 19c shows a splice in the sill of a trestle bent, in which the fish plates have only two tables. The sill is subject to only a small tensile stress, due to the inclination of the batter posts, its main function being to act as a beam to distribute the load from the posts to the foundations.

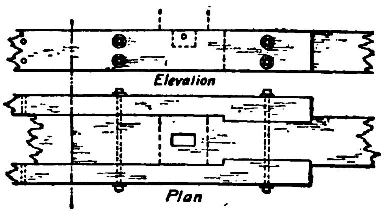


FIG. 19c. Splice in Sill of Trestle Bent.

In Fig. 19d is indicated a splice in the lower chord of a roof truss of an exposition building. The two white pine timbers break joints so that only one needs to be spliced at any given section. The oak fish plate also acts as a packing block. Since

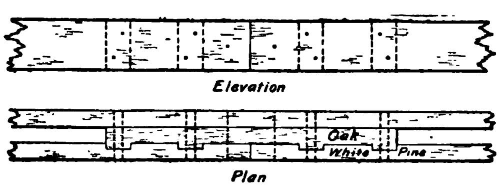


FIG. 19d. Splice in Lower Chord of Truss.

there are tables on only one side of the timber, flexure is developed which causes additional stresses in the fish plate and other chord timber. The shorter tables on the fish plate indicate the greater resistance of oak to shearing along the grain. Another example may be found in Engineering Record, vol. 44, page 127, Aug. 10, 1901.

Sometimes the tabled fish-plate joint is modified by cutting notches opposite to each other in both the fish plates and main timber and inserting transverse keys. An example of this kind

may be seen in Fig. 69e and in Engineering News, vol. 24, page 276, Sept. 27, 1890, in a splice of the sill of a falsework trestle bent. It is less efficient than the regular form on account of the tendency of the keys to rotate and the smaller resistance to compression on the side of the fibers.

The indented fish-plate joint in Fig. 19e is shorter than the tabled joint with the same number of tables, but requires more careful workmanship and very straight-grained timber. Although

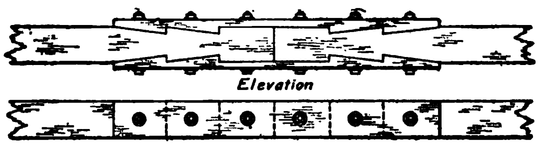


FIG. 19e. Indented Fish-plate Joint.

the shearing surface of the table extends, theoretically, over its full length, it cannot be so regarded practically on account of the tendency to inclined shear near the thin edge of the table. Another advantage lies in the fact that the bolts are neither in the shallowest section of the main timber nor of the fish plates.

Prob. 19. Compute the approximate relation between the lengths of the tables in Fig. 19d, provided the working unit stresses adopted by the American Railway Engineering and M. of W. Association are specified (Art. 82).

ART. 20. DESIGN OF TABLED FISH-PLATE JOINT.

The following order of design is suggested as a guide to the student who is just beginning to apply the principles of mechanics to the design of structures. Its aid simplifies the computations, avoids unnecessary revisions, and secures a systematic arrangement of the work.

ORDER OF DESIGN: 1. Compute the required areas for tension, compression, and shear. 2. Assume the number of tables on each of the main timbers. Usually four tables give the best results. 3. Determine the width of the timber and the

approximate length of the tables. 4. Assume the diameter of the bolts after observing those on similar designs, and find the net and gross depth of the main timbers. 5. In case different material is used for the fish plates some of the computations indicated above are to be repeated for the fish plates. 6. Find the required net section area of the bolts, as well as the required bearing area and diameter of the washers. Consult the tables of standard bolts and washers. If no standard washer meets the requirements, design one for the given case. Record the net areas furnished by the bolts and washers adopted. 7. Compare the computed and assumed diameters of the bolts and revise the gross section of the timber if necessary. 8. If the gross depth of the timber adopted exceeds that required, decide whether to include the excess either in the net depth or in the height of tables, or to divide the excess so as to increase both the tensile and compressive areas. State the reason governing the decision. 9. Make an alternative design by taking a different width to learn whether a more economical section of the timber may be secured, or the bolts may be reduced in size or length. 10. Prepare a bill of material. 11. Make an estimate of cost. 12. Make a working drawing of the joint, including a side elevation and a plan drawn to a scale of $1\frac{1}{2}$ inches to the foot.

For example, let it be required to design a tabled fish-plate joint to transmit a tension of 64 000 pounds. The timber to be used is longleaf yellow pine. Let it also be assumed that the timber is carefully selected for the purpose and that the joint is in a building protected from the weather, as in the lower chord of a roof truss. Let the working unit stresses for the timber be specified as follows: For tension, 1800 pounds per square inch; compression with the grain, end bearings, 2000 pounds; compression across the grain when under washers 550 pounds; shear along the grain, 250 pounds. Let the unit tensile stress in the bolts be taken at 15 000 pounds per square inch. Remembering the principles given in Art. 19 and the suggested order of design,

the following computations are made: The net tensile area is $64\,000/1800 = 35.6$ square inches, the compressive area is $64\,000/32.0$ square inches, and the shearing area is $64\,000/250 = 256.0$ square inches. If four tables be adopted for each timber, the shearing area of one table is $256.0/4 = 64.0$ square inches. In order that the length of the table shall not be less than two-thirds of its width nor materially to exceed its width, the width of the timber may be either 8 or 9 inches. Using the former value, the length of the table must slightly exceed $64/8 = 8$ inches in order to allow for two bolt holes.

Let the diameter of the bolts be assumed temporarily as $\frac{3}{4}$ inch, while the diameter of the bolt holes is made $\frac{1}{8}$ inch larger. The net width of the tensile section is therefore $8-(2\times\frac{7}{8}) = 6.25$ inches and the net depth of the main timber is $35.6/6.25 = 5.70$ inches, or when expressed in carpenter's measure, $5\frac{3}{4}$ inches. The depth of the tables must be $32.0/(4 \times 8) = 1$ inch in order to furnish the necessary bearing area. This makes the gross depth of the timber $5.75 + (2 \times 1) = 7.75$, or 8 inches, since the former value is not a standard dimension.

Since the same kind of wood is used for the fish plate as for the main timbers, the stress in each plate is one-half of that transmitted by the joint, and hence both its net and gross depths are one-half of those of the main timber. Considering only the stress carried by one table of the fish plate, the lever arm of the couple composed of the resultants of the tension in the net section of the fish plate and of the corresponding compression on the end of the table is one-half of the gross depth of the fish plate, and therefore the moment of rotation of this couple is $(\frac{1}{4} \times 64\,000)\ (\frac{1}{2} \times 4) = 32\,000$ pound-inches. The width of the timber requires two bolts at each table (Art. 19). The lever arm of the resultant pressure under the washer for each bolt about the center of rotation in the end of the table is at least one-half of the length of the table, or $8/2 = 4$ inches. Hence the stress in one

bolt is $\frac{1}{2}$ (32 000)/4 = 4000 pounds, and the required area of the bolt at the root of the thread is 4000/15 000 = 0.267 square inch. On reference to the manufacturer's handbook it is found that the diameter of the bolt needed is only $\frac{3}{4}$ inch, and its net section area 0.302 square inch. As this agrees with the size assumed, no revision of the net depth is necessary. The section area of a $\frac{7}{8}$-inch hole is 0.601 square inch, and by adding two such areas to the required shearing area of the table its length is found to be 65.202/8 = 8.15 or $8\frac{1}{4}$ inches.

The required bearing area of the washer is 4000/550 = 7.27 square inches, and the area of its $\frac{7}{8}$-inch hole is 0.60 square inch. By reference to a handbook, the diameter of a circle having an area not less than 7.87 square inches, is found to be $3\frac{3}{16}$ inches. On consulting manufacturer's list or tables of standard washers it is found to be necessary to select one with a diameter of $3\frac{1}{4}$ inches.

If the excess of $\frac{1}{4}$ inch in depth be divided between the tensile and compressive areas, it will increase the net depth to $5\frac{7}{8}$ and the height of tables to $1\frac{1}{16}$ inches, but as the timber dimensions are all to be expressed in full eighths of an inch, it is probably best to make the net depth of the main timber 6 inches, and leave the tables one inch high. If the fish plates are extended $1\frac{1}{2}$ inches beyond the end tables, the total length of each plate is $8 \times 8\frac{1}{4} + 2 \times 1\frac{1}{2} = 69$ inches = 5 feet 9 inches.

If, on the other hand, a width of 9 inches be taken instead of 8 inches, the length of table becomes $7\frac{1}{4}$ inches, the net depth 5 inches, and the height of table 1 inch, while the diameter of the bolts remains the same as before. Unless the timber is sawed to order, the size of 8 by 8 is preferable to that of 7 by 9 inches. It is also found that no improvement can be made by changing the number of tables to two or six. For six tables the main timber becomes 7 by 8 inches, but the saving in timber is probably insufficient to counterbalance the increased cost of the bolts

and workmanship. An accurate comparison of cost can only be made by taking into account the length of the main timbers which are to be spliced in any actual case.

A third design may be computed for a width of 10 inches. In this case the length of table is $6\frac{5}{8}$ inches, the net depth $4\frac{1}{4}$ inches, the height of table $\frac{7}{8}$ inch, the gross depth 6 inches, the diameter of bolts $\frac{5}{8}$ inch, and of the washers $2\frac{3}{4}$ inches. It is necessary, however, to move the bolts a little so as to give their stress a lever arm of 4 inches, in order to reduce their diameter to $\frac{5}{8}$ inch. If $\frac{3}{4}$-inch bolts were used, it would be necessary to make the tables $\frac{13}{16}$ inch high in order not to exceed the gross depth of 6 inches. This height gives sufficient bearing area on the end of each table. The use of the $\frac{7}{8}$-inch tables and $\frac{5}{8}$-inch bolts is preferable. The total length of each fish-plate is 56 inches. By selecting this design rather than either of the others, a small saving of material is effected, the labor for construction being the same.

The detail drawing for the first design, with all the dimensions required for its construction, is given in Fig. 20*a*. The bill of material for the joint and the main timbers, which are assumed to be 20 feet long, is as follows:

2 main timbers, 8″ × 8″ × 20′ 0″	213.3 ft. B.M.
2 fish plates, 4″ × 8″ × 5′ 9″ [6′ 0″]	32.0 ft. B.M.
10 $\frac{3}{4}$″-bolts, $16\frac{1}{2}$″ long; 10 nuts	23.9 lb.
20 O.G. washers, $3\frac{1}{4}$″ diam. @ 1.0 lb.	20.0 lb.

As the commercial lengths are multiples of 2 feet, it is necessary to compute the contents of the fish plates for a length of 6 feet.

The estimate of cost is as follows:

246 ft. B.M. lumber @ 3¢	=	$7.38
24 lb. bolts and nuts @ 3¢	=	0.72
20 lb. washers @ $2\frac{1}{2}$¢	=	0.50
8 hr. labor @ 35¢	=	2.80
Total cost for design No. 1	=	$11.40

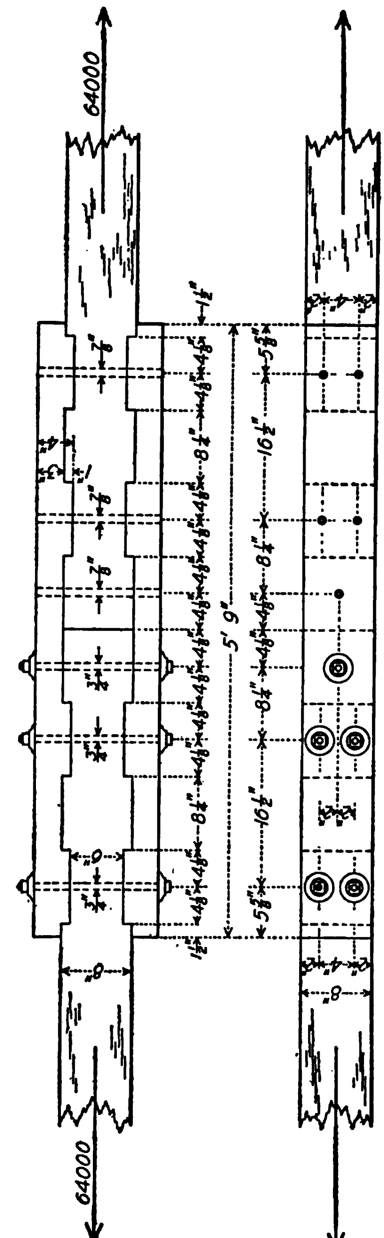


FIG. 20a. Working Drawing of Tabled Fish-plate Joint.

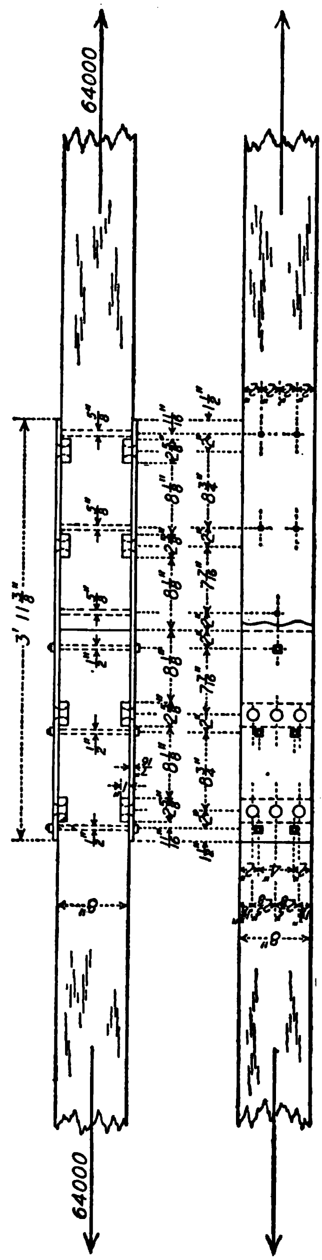


FIG. 20b. Working Drawing of Joint with Steel Tabled Fish Plates.

The cost for designs Nos. 2 and 3 are, respectively, $11.18 and $10.32, and therefore the last design should be adopted.

Prob. 20. Design a tabled fish-plate joint of longleaf yellow pine to transmit a tensile stress of 32 000 pounds.

ART. 21. TABLED FISH PLATES OF STEEL.

Fig. 21*a* shows a splice in one of the three timbers composing the lower chord of a roof truss, in which tabled fish plates of steel are employed. The tables on the fish plates consist of 'flats,' which are riveted to the plates. These tables are often called 'ribs.' The heads of the rivets are countersunk in the flats to avoid further reduction of the net tensile section of the timber.

An illustration of the use of fish-plate joints of this type may be seen on the inset accompanying an article on the transporta-

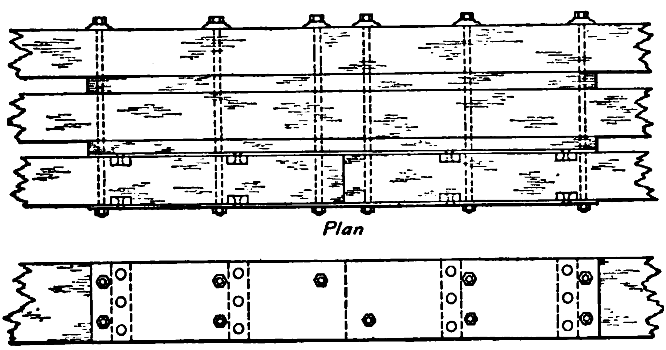


FIG. 21*a*. Splice in Lower Chord with Steel Tabled Fish Plates.

tion building of the World's Columbian Exposition in Engineering News, vol. 28, page 605, December 29, 1892. The plans of the lower chords of two of the roof trusses show how the different timbers which compose the chords break joints.

In designing the fish plates it is necessary to observe the standard specifications for steel structures, which are based on experience. The spacing of rivets, center to center, in the

direction of the row, is known as the pitch. The minimum allowable pitch is three diameters of the rivet, but it is generally preferable to use a slightly larger value. The minimum distance from the center of a rivet to the rolled edge of a plate, or of a flat, is $1\frac{1}{2}$ and preferably $1\frac{3}{4}$ diameters. The best sizes of rivets for such a joint are $\frac{5}{8}$, $\frac{3}{4}$, and $\frac{7}{8}$ inch in diameter. The standard sizes and proportions of rivets are given in the handbooks of various bridge manufacturers. The minimum thickness of plates should be $\frac{1}{4}$ inch when not exposed to the weather, and $\frac{5}{16}$ inch when so exposed. A limit is fixed on account of the deterioration of the metal, due to rust or corrosive gases. Where corrosion, due to gas, is unusual, an extra addition is made to the thickness of the metal.

In staggered riveting the distance between the rows should not be less than one-half of the pitch of the rivets in a row, plus one-quarter of the diameter of a rivet hole.

Another rule prescribes that this distance shall not be less than 2 diameters of the rivet for staggered riveting, while $2\frac{1}{2}$ diameters are to be used for chain riveting.

The distance from the rivet hole to the end of a tension plate should not be less than 80 percent of the net distance between the rivet holes in a row, or the distance from the center line of rivets to the end of the plate should not be less than $1\frac{3}{4}$ diameters of the rivet.

In chain riveting, the rivets in adjacent rows are directly opposite each other, while in staggered riveting the rivets in one row are located in intermediate positions with respect to those in the other row.

In designing the fish plates the ribs may be regarded as beams under a uniform load, due to the compression from the ends of the wooden fibers, and with supports at the rivets. Usually, however, the minimum allowable width gives sufficient strength in this regard. The rivets are subject to single shear, and also

to bearing both in the rib and the plate. The bearing area of a rivet is the projection of its cylindrical bearing surface upon a diametral plane, and equals the product of the rivet diameter by the thickness of the plate. The thinnest plate through which any rivet passes determines its bearing value. The balance of the design is practically the same as that explained in Arts. 19 and 20. It may be added, however, since the bolts are so close to the steel ribs, that the net section of the timber should be taken the same as if the net depth and the net width occurred in the same cross-section.

Prob. 21. Make a list of the tabled fish-plate joints, with metal fish plates, shown in the illustrations in Chaps. IV and V.

ART. 22. DESIGN OF TABLED FISH PLATES OF STEEL.

Let it be required to design a tabled fish-plate joint with steel fish plates for a tensile stress of 64 000 pounds in order to compare it with the one designed in Art. 20, which has wooden fish plates. The tensile, compressive, and shearing areas for the wood are 36.6, 32.0, and 256.0 square inches respectively, being the same as found in Art. 20. Let the same width be taken as before, or 8 inches for the first design.

The unit stress to be used for tension in the steel plate and bolts is 15 000 pounds per square inch, while those for shear and bearing of the rivets are 10 000 and 20 000 pounds per square inch respectively. With the aid of the rivet table in a handbook the values in single shear for rivets $\frac{3}{4}$ and $\frac{7}{8}$ inch are found to be 4418 and 6013 pounds. As four tables will again be adopted, the pressure on one rib or table of the fish plate is 16 000 pounds, which requires either four $\frac{3}{4}$-inch rivets or three $\frac{7}{8}$-inch rivets. Four rivets cannot be used, since it will make the rivet pitch $8/4=2$ inches, which is less than the allowable value of three times the rivet diameter. Accordingly, it is necessary to use three $\frac{7}{8}$-inch rivets, spacing them with a pitch of $2\frac{5}{8}$ inches, the outside ones being placed $1\frac{3}{8}$ inches from the edge of the plate.

The net tensile section of the plate required is 64 000/(2 × 15 000) = 2.133 square inches. The net width of the plate is 8 − 3 × 1 = 5 inches, since specifications generally require a deduction for holes $\frac{1}{8}$ inch larger than the diameter of the rivets, in all tension members. The thickness of the fish plate is 2.133/5 = 0.427 or $\frac{7}{16}$ inch. As the bearing of a $\frac{7}{8}$-inch rivet in a $\frac{7}{16}$-inch plate is 7656 pounds (see handbook) for the specified unit stress, the rivets are also safe in bearing, and no revision is necessary.

According to the practical requirements given in Art. 21, the rib must be at least $2 \times 1\frac{1}{2} \times \frac{7}{8} = 2\frac{5}{8}$ inches wide. The thickness of the rib is 1 inch, or the same as the height of table for the wooden fish plate. On consulting a manufacturer's handbook it is found that a flat $2\frac{5}{8}$ by 1 inches is a commercial size, and may therefore be adopted. Since the flat is thicker than the diameter of the rivets, the holes should be drilled.

Let the centers of bolts be placed 2 inches from the center line of the rivets or of the rib, making the lever arm for the tension in the bolts $2 + \frac{1}{2}(2.625) = 3.312$ inches. The moment of rotation for the rib is $16\,000 \times \frac{1}{2}(1.0 + 0.438) = 11\,500$ pound-inches. The tension in each of the two bolts is $11\,500/(2 \times 3.312) = 1736$ pounds, and the area required at the root of the thread is $1736/15\,000 = 0.116$ square inches. This requires a $\frac{1}{2}$-inch bolt, which has a net area of 0.126 square inch. The bolt holes should be $\frac{1}{16}$ inch larger than the bolts, and the distance from their centers to the end of the fish plate is made $1\frac{1}{2}$ inches (Art. 21).

The net width of the timber is $8 - 2 \times \frac{5}{8} = 6.75$ inches, and the net depth is therefore $35.6/6.75 = 5.274$, or $5\frac{3}{8}$ inches, making the gross depth $5\frac{3}{8} + 2 \times 1 = 7\frac{3}{8}$ inches, the same as in Art. 50. The length of the table is $(64.0 + 2 \times 0.307)/8 = 8.08$, or $8\frac{1}{8}$ inches, since the area of a $\frac{5}{8}$-inch hole is 0.307 square inches. The total length of the fish plate is $2(2 \times 8\frac{1}{8} + 1\frac{1}{2} \times 2\frac{5}{8} + 2 + 1\frac{1}{2}) = 47\frac{3}{8}$ inches.

On making an alternative design for a width of 10 inches, the following results are obtained: The steel fish plates are $\frac{3}{8}$ inch in thickness; four $\frac{3}{4}$-inch rivets are required in each rib; the ribs are $2\frac{1}{2}$ by $\frac{13}{16}$-inch flats; the bolts are $\frac{1}{2}$ inch in diameter, and placed $1\frac{3}{4}$ inches from the center line of the ribs; the required net depth is $4\frac{1}{8}$ inches, but as the gross depth is 6 inches, the actual net depth remaining is $4\frac{3}{8}$ inches; the length of each wooden table is $6\frac{1}{2}$ inches; and the total length of each fish plate is 40 inches. An examination of these dimensions indicates a small saving of material in both timber and steel over that for the first design.

The detail drawing for the first design, with all the necessary dimensions, is given in Fig. 20*b*. The bill of material for the joint and the main timbers, assumed to be 20 feet long, is as follows:

2 main timbers, $8'' \times 8'' \times 20'\ 0''$	213.3 ft. B.M.
2 steel plates, $8'' \times \frac{7}{16}'' \times 3'\ 11\frac{3}{8}''$	94.0 lbs.
8 flats, $2\frac{5}{8}'' \times 1'' \times 8''$	47.6 lbs.
10 $\frac{1}{2}''$-bolts, $9\frac{1}{2}''$ long; 10 nuts	6.4 lbs.

The estimate of cost is as follows:

214 ft. B.M. lumber @ 3¢	$6.42
142 lb. steel plates (riveted) @ 4¢	5.68
7 lb. bolts and nuts @ 3¢	0.21
5 hr. labor @ 35¢	1.75
Total cost for first design	$14.06

In a similar manner the cost of the alternative design is found to be $13.17, and therefore the alternative design should be adopted.

Prob. 22. Compute the working strength of the joint in the lower chord of the upper roof truss shown on Plate V.

ART. 23. PRESSURE OF WOOD ON METAL PINS.

In framing, it is sometimes necessary to use metal pins or bolts as beams in transferring stresses from one timber to another. This involves the determination of the pressure

of the fibers of the wood upon the cylindrical surface of the pin. When the resultant of the pressure is perpendicular to the fibers of the wood, the magnitude of the resultant is the same as if the bearing surface were the diametral section of the pin. But when the direction of the resultant is parallel to the fibers of the wood, the case is entirely different because the resistance of the fibers to lateral compression is much less than to longitudinal compression.

Theoretically, the resultant pressure may be determined approximately in the following manner: Let S be the longitudinal unit pressure which is parallel to the fiber, p the unit pressure normal to the surface of the pin, θ the angle which it makes with

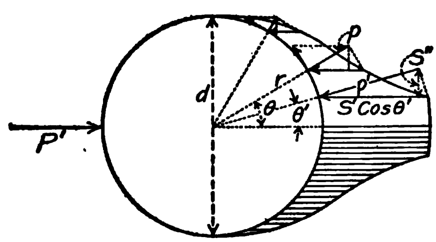


FIG. 23*a*.

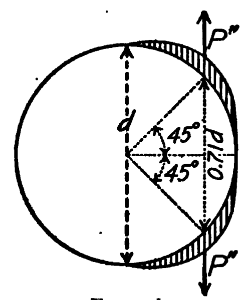


FIG. 23*b*.

the direction of the fibers, as indicated in Fig. 23*a*, r the radius and d the diameter of the pin or bolt, and h the height of the timber bearing against the pin. Let S' be the safe unit-stress for compression parallel to the fiber, that is, for bearing on the ends of the fibers, and S'' for bearing on the side of the fibers.

The normal unit pressure p is the component of the longitudinal unit pressure S, or $p = S \cos\theta$. It may be assumed that S equals S' for values of θ from 0 to θ', the latter angle being that for which the transverse component of p equals S''. Accordingly, $S'' = S' \cos\theta' \sin\theta'$ or $\sin 2\theta' = 2\, S''/S'$. The transverse component of p must then gradually diminish until it becomes zero when $\theta = 90°$. Let it be assumed to vary directly as $\cos\theta$.

Its value is then $S'' \cos\theta/\cos\theta'$, from which is found $p = S'' \cos\theta/\cos\theta' \sin\theta$ and $S = S''/\cos\theta' \sin\theta = S' \sin\theta'/\sin\theta$, between the limits of θ' and $90°$.

Let the coördinate y of any point on the circumference be perpendicular to the fibers of the wood, then $y = r \sin\theta$ and $dy = r \cos\theta\, d\theta$. The pressure on the area hdy is $Shdy$, and by integrating this expression between the limits of $+90°$ and $-90°$, the resultant longitudinal pressure on the pin is found to be

$$P' = hdS' \int_0^{\theta'} \cos\theta\, d\theta + hd\, S' \sin\theta' \int_{\theta'}^{90} \cot\theta d\theta$$
$$= hdS'[\sin\theta'(1 - \log_e \sin\theta')]. \qquad (1)$$

For wood having a ratio of 0.25 between the safe unit bearing on the side and on the end of the fibers respectively, $\theta' = 15°$, and

$$P = 0.62\ hdS'. \qquad (2)$$

This result indicates that the safe longitudinal pressure of such timber on the side of a round pin is approximately six-tenths of the corresponding pressure on a square pin of the same diameter. When the ratio referred to is reduced to 0.20, the numerical coefficient in (2) becomes 0.53.

An experimental determination for longleaf yellow pine, but in which the timber was tested to its ultimate strength, gave an average coefficient of 0.63 for 5 tests. The specimens were prevented from splitting by means of clamps. The plane of division between the fibers crushed sidewise was marked in every case and gave an average value for θ' of $15\frac{1}{2}°$.

The transverse components of the normal pressures on each half of the pin are respectively equal and opposite to those on the other side, and together they tend to split the timber (Fig. 23*b*). The transverse component of P is $S' \cos\theta \sin\theta$ for values of θ between 0 and θ', and is $S'' \cos\theta/\cos\theta' = S' \cos\theta' \sin\theta' \cos\theta$ between θ' and $90°$. Let the coördinate x of any point on

the circumference be parallel to the fibers, then $x = r\cos\theta$ and $dx = -r\sin\theta\, d\theta$. The negative sign may be neglected since it is immaterial whether the horizontal side of the rectangle hdx is measured in a positive or negative direction to obtain its area. The resultant transverse pressure on the pin is

$$P'' = \tfrac{1}{2}\, hdS' \int_0^{\theta'} \cos\theta \sin^2\theta \, d\theta + \tfrac{1}{2}\, hdS' \cos\theta' \sin\theta' \int_{\theta'}^{90^\circ} \cos\theta \sin\theta \, d\theta$$
$$= \tfrac{1}{2}\, hdS' (\tfrac{1}{3} \sin^3\theta' + \tfrac{1}{4} \sin 2\theta' \cos^2\theta'). \qquad (3)$$

For wood in which the ratio of S'' to S' is 0.25, equation (3) becomes $P'' = 0.061\, hdS'$; and substituting the value of S' from (2) there is obtained the relation

$$P'' = 0.10\, P'. \qquad (4)$$

In other words, the transverse stress which must be resisted by tension across the fiber is approximately one-tenth of the longitudinal pressure on the pin. For a ratio of $S''/S' = 0.20$ the

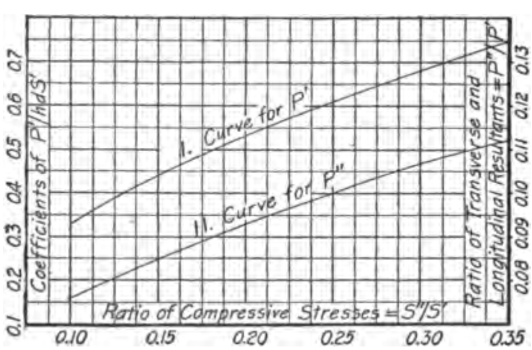


FIG. 23c.

coefficient in (4) reduces to 0.093. It may be of interest to add, that the resultant P'' is applied at a point for which θ is about 45 degrees. Fig. 23c gives two curves from which the numerical coefficients in equations (2) and (4) may be found directly, when the allowable unit-stresses S'' and S' have been determined for the given species of wood to be used in the construction.

When the resultant of the pressure of the wood on a round pin is perpendicular to the fibers, the magnitude of the safe bearing value is to be taken as hdS''; that is, the pressure is the same as if the pin were square or rectangular in cross-section.

Prob. 23. If the allowable unit compressive stresses on the side and ends of the fibers of white oak are respectively 450 and 1300 pounds per square inch, it is required to compute the values of P' and P''.

ART. 24. PLAIN FISH-PLATE JOINT.

In this joint the main timbers are spliced end to end by means of two plain fish plates and connecting bolts, as shown in Fig. 24*a*. The stress is transmitted from each main timber to the fish plates through the bolts acting as beams.

The stress is first carried by the continuous fibers past the bolts, and then transferred to the fibers directly behind the bolts, through their lateral cohesion or the shearing strength

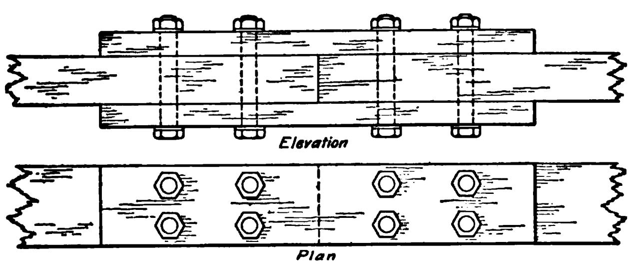


FIG. 24*a*. Plain Fish-plate Joint.

parallel to the fibers. The pressure of the ends of these fibers constitutes the load on the bolt beam, while the reactions consist of the corresponding pressures of the fish plates.

Theoretically, the best arrangement would be to use square bolts with their sides respectively parallel and perpendicular to the fibers of the wood, since the flexural strength of a square bolt is 67 percent larger than that of a round bolt of the same diameter, while the corresponding effective bearing areas of the wood may differ by a percentage ranging from about 25 to

over 60. Practical difficulties in construction, which materially increase the cost, have hitherto prevented the adoption of square bolts.

In the design of the plain fish-plate joint the following facts and principles must be kept in mind: First, the reduction in the effective bearing area of the wood against a bolt, on account of the cylindrical surface of the latter, as discussed in the preceding article. In general, an increase in the compressive area requires an increase in the number of bolts, but occasionally a slight increase may be obtained by increasing the diameter of the bolts. Second, the wood tends to shear in two surfaces not tangent to the bolt, but closer together on account of the unequal pressure around the surface of the bolt. In spacing the bolts center to center longitudinally, it is best to add the full diameter to the length of the shearing surfaces required. Third, the transverse components of the pressure on the bolts tends to split the timber at or near the axial plane of the bolts. As this surface is so close to the shearing surface they naturally tend to combine, especially if the timber is not entirely straight-grained. It is on the side of safety, therefore, to add the required length of the splitting surface to that of each shearing surface, since it may combine with either one. The necessity for this was shown by some experiments made by ELMER ZARBELL in 1895. Sometimes small transverse bolts are inserted to resist the tendency to split the timber. Their use shortens the length of the joint, but increases its cross-section. Fourth, on account of the low shearing strength of wood parallel to the grain, the bolts are placed directly opposite when more than one row is used with wooden fish plates.

Fifth, according to mechanics, the distribution of the bending moments in a bolt is that illustrated in Fig. 24*b*, when it is assumed that the pressure on the bolt is uniformly distributed along its length, as indicated by the full arrows. The curves

ac, *cg*, and *gi* are parabolas. When the pressures in each fish plate, and in each half of the main timber, are replaced by their respective resultants, as shown by the broken arrows, the moment diagram becomes the polygon *bdfh*, in which the maximum ordinate is the same as before. Accordingly, the maximum bending moment equals the stress in the fish plate multiplied by the distance from the center of the fish plate to the quarter point

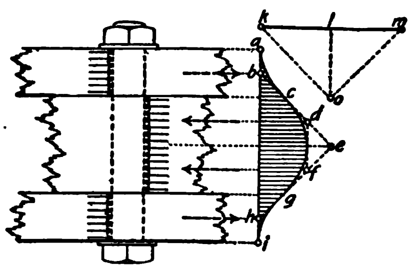


FIG. 24*b*. Bending Moments in Bolt.

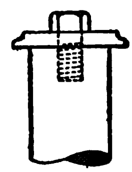


FIG. 24*c*.

of the main timber. Since the bending moment varies directly as the depth of the timber, it is desirable to keep the depth as small as practicable while maintaining a fair relation between the depth and width. If the fish plate is too wide in proportion to its thickness, it is more liable to warp. The resisting moment of a bolt equals $S\pi d^3/32$, in which S is the stress in the outer fiber and d the diameter. The strength of the bolt, therefore, varies as the cube of the diameter, and for a given change in resisting moment the number of bolts changes rapidly in proportion to any change in diameter.

In the plain fish-plate joint the bolts are not subject to any appreciable tension which requires consideration in designing, since the nuts are only drawn up lightly to bring the fish plates into contact with the main timbers. On this account it is not necessary for the bolts to have the standard size nut and head of common bolts. The joint will also appear clumsy if these standards are used. For this purpose lateral pins used in

bridges, but with shallow nuts instead of cotters, are well adapted, the standard sizes of which are given in Cambria Steel and in the handbooks of some other steel manufacturers. When wooden fish plates are used so that the bearing is against the wood only, the nominal diameter may be used, thus avoiding the necessity of turning down the pin to the finished diameter as required for bearing against steel. Another arrangement is to insert a tap bolt and washer at each end of the bolt as indicated in Fig. *24c*.

In some cases it may be a more economical arrangement to employ double extra hydraulic tubing for the pins, and use small bolts and washers to hold the parts securely in place. A tube with outside and inside diameters of 1.31 and 0.58 inches respectively has 96.1 percent of the flexural strength of a solid bolt of the same external diameter. For corresponding diameters of 1.66 and 0.88 inches the percentage is 92.2; for diameters of 1.90 and 1.08 inches the percentage is 89.5; and for diameters of 2.37 and 1.49 inches the percentage is 84.5.

Since the preceding paragraphs were written, a new and more effective device for boring square holes in wood or metal has been introduced in this country. When such a device is available so that the square holes may be bored accurately as well as economically and with a finish on the walls of the hole equal to that produced by a twist drill, the square bolt should be used in plain fish plate joints, thus reducing the size of bolts and eliminating the tendency to split the timber. See article on A Drill and Chuck for boring Square Holes, in Engineering News, vol. 61, page 24, Jan. 7, 1909.

The application of plain fish-plate joints is shown in the plans of a coal pocket in Engineering Record, vol. 33, page 192, Feb. 15, 1896, and in the falseworks for erecting a bridge, on the inset of Engineering News, vol. 43, page 164, March 8, 1900.

For compression members this form is preferred to the tabled fish-plate joint. In this case also the fish plates are put on the

four sides of the stick as shown in Fig. 24*d*, and the bolts are much smaller than for a tension splice, the object of the fish

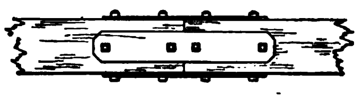


FIG. 24*d*. Splice in Post.

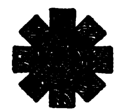


FIG. 24*e*.

plates being merely to prevent deflection in the column and not to take any part of the direct compression. In Fig. 24*f* is shown a plain fish-plate splice of unusual length used in forming piles 130 feet long, to support certain parts of the falsework of the Poughkeepsie bridge and in connection with the anchorage of

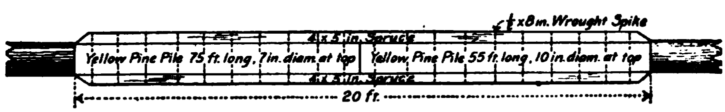


FIG. 24*f*. Fish-plate Joint in Long Pile.

the cribs during the sinking process. An enlarged section is shown in Fig. 24*e*. See Engineering News, vol. 17, page 410, June 25, 1887.

Prob. 24. Compute the strength of the joint at the center of the lower chord in Fig. 69*c*, assuming unit-stresses per square inch for the wood of 1300 pounds for tension, 1100 and 275 pounds for compression on the ends and sides of the fibers respectively, and 130 pounds for shear parallel to the fiber; and 15 000 pounds for the bolts in flexure.

ART. 25. DESIGN OF PLAIN FISH-PLATE JOINT.

The following order is suggested for the computations needed to make the design: 1. Compute the required net area in tension, the area in compression for square bolts, the equivalent compressive area for round bolts, the shearing area, and the longitudinal area to resist splitting. 2. Assume the depth, observing that the bolts make a large difference between the gross and net width of the cross-section. 3. Compute the bending moment to be resisted by the bolts in one-half of the joint.

4. Assume the number of bolts, the number being an even one when two rows of bolts and wooden fish plates are used. 5. Determine the diameter of the bolts, and check the diameter by means of the required compressive area. 6. Find the net and gross widths of the timber. 7. Compute the longitudinal distance between centers of bolts, the distance from center of bolt to end of timber, and the total length of the joint. 8. Try a different depth to learn whether better results can be secured.

For example, let the joint be designed to resist the same tension as the tabled fish-plate joint in Art. 20, using the same kind of wood. Besides the safe unit-stresses given in that article, the following are required: for tension across the grain of the timber, 150 pounds per square inch, and for the stress in the outer fiber of the bolt or pin under flexure, 24 000 pounds per square inch.

In this example, let it be assumed that the ratio between the allowable compression on the side of the fiber to that on the end of the fiber is 0.25 (Art. 23). The net tensile area is $64\,000/1800 = 35.6$ square inches; the compressive area for square bolts is $64\,000/2000 = 32.0$ square inches; and for round bolts, $32.0/0.6 = 53.3$ square inches; the shearing area is $64\,000/250 = 256.0$ square inches; and the splitting area $0.1 \times 64\,000/150 = 42.7$ square inches.

Assuming a depth of 5 inches, the bending moment is $32\,000 \times 2.5 = 80\,000$ pound-inches. If 6 bolts be used, the resisting moment of each bolt is 13 330 pound-inches, which, according to a table of bending moments on pins, requires a diameter of $1\frac{1}{1}\frac{8}{6}$ inches. The next larger commercial size of lateral pins (see Cambria Steel) is $1\frac{15}{16}$ inches, which will be adopted. The compressive area is then $6 \times 1\frac{15}{16} \times 5 = 58.1$ square inches, indicating that the number of bolts was correctly assumed. The net width of the cross-section is $35.6/5 = 7.12$

inches, and the gross width $7.12 + 2 \times 1.9375 = 11$ inches. The length of the two shearing surfaces behind each bolt is $256/(2 \times 6 \times 5) = 4.27$ inches, and of the splitting surface, $42.7(6 \times 5) = 1.42$ inches. The longitudinal spacing of the bolts is therefore $4.27 + 1.42 + 1.94 = 7.63$ or $7\frac{5}{8}$ inches between centers, while the distance from center of bolt to end of timber is 0.97 inch less, or $6\frac{3}{4}$. The length of the fish plate is $2(2 \times 7\frac{5}{8} + 2 \times 6\frac{3}{4}) = 57.5$ inches or 4 feet $9\frac{1}{2}$ inches.

Let an alternative design be made by assuming a depth of 6 inches for the main timber. The bending moment is then $32\,000 \times 3 = 96\,000$ pound-inches, which can also be resisted by 6 bolts $1\frac{1}{16}$ inches in diameter, since the resisting moment of

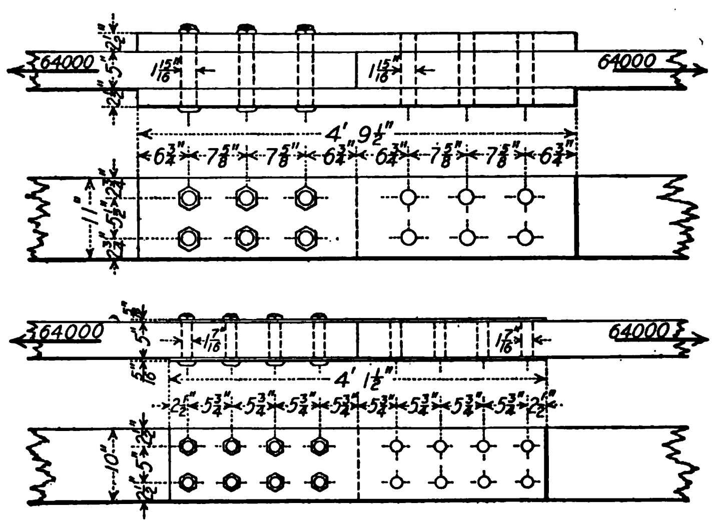


FIG. 25*a*. Working Drawing of Plain Fish-plate Joint.
FIG. 25*b*. Working Drawing of Joint with Steel Fish Plates.

such a bolt is 17 200 pound-inches, for the specified unit-stress. The compressive area is $6 \times 1\frac{15}{16} \times 6 = 69.75$ square inches. The gross width is found to be 10 inches, the spacing of the bolts between centers is $6\frac{3}{4}$ inches, and from center of bolt to end of timber $5\frac{3}{4}$ inches. The total length is 50 inches. The

5 by 11-inch timber required for the first design is not usually kept in stock, and hence the 6 by 10-inch timber of the alternative design is selected. The latter, however, requires the bolts to be one inch longer.

For very large stresses the smallest number of bolts which gives the necessary compressive area may require them to be spaced so far apart longitudinally that the transverse tension cannot be properly distributed. It must be remembered that the components of the pressure on each bolt which tends to split the timber is acting at the bolt, and therefore the longitudinal surface resisting this tendency should not extend too far beyond the bolt. It may hence be better in such cases to increase the number of bolts in order to reduce the combined length of the splitting and shearing surface behind each bolt.

The detail drawing for the alternative design with all the dimensions required for its construction is given in Fig. 25*a*. The bill of material for the joint and main timbers, assumed to be 20 feet long, is as follows:

2 main timbers, 6″ × 10″ × 20′ 0″	200.0 ft. B.M.
2 fish plates, 3″ × 10″ × 4′ 2″ [4′ 6″]	22.5 ft. B.M.
12 lateral pins, $1\frac{1}{8}$″ diam., $12\frac{1}{2}$″ long, with nuts,	148.8 lb.

In computing the contents of the fish plates it is assumed that four pieces are cut from an 18-foot length. The estimate of cost is as follows:

223 ft. B.M. lumber @ 3¢	$6.69
149 lb. lateral pins @ 10¢	14.90
3 hr. labor @ 35¢	1.05
Total cost for alternative design	$22.64

In a similar manner the cost of the first design is found to be $21.13, and if the timbers were sawed to order, this design would be selected. The cost may be reduced by methods explained in the next article.

Prob. 25. Design a plain fish-plate joint of longleaf yellow pine to transmit a tensile stress of 96 000 pounds.

ART. 26. PLAIN FISH PLATES OF STEEL.

The use of steel fish plates reduces the length and diameter of the bolts, since the bending moment is less than with wooden fish plates. In designing the fish plate the thickness must be sufficient to provide the necessary bearing area on the bolts. The width is determined by the net tensile area, but as it cannot exceed the width of the timber, the thickness may have to be increased. The plate must be extended far enough beyond each end bolt to act as a beam in order to transfer the pressure or load from the bolt to the net tensile sections on each side of the bolt.

As indicated in Fig. 26*a* an odd number of bolts may be used in each half of the joint, but the longitudinal spacing should be

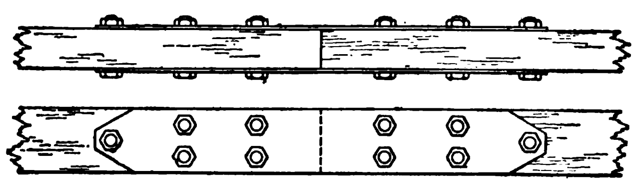


Fig. 26*a*. Joint with Plain Fish Plates of Steel.

the same as if the number were even. In this case with five bolts, it will be observed that for the timber the critical section in tension is not at the end bolt but at the next pair of bolts, since that section transmits four-fifths of the entire stress in the joint.

Let a design be made for a plain fish-plate joint carrying the same stress as that in Art. 25 but with fish plates of steel instead of wood. Accordingly the tensile, compressive, shearing, and splitting areas for the timber remain the same as before; namely, 35.6, 53.3, 256.0, and 42.7 square inches respectively. The unit-stresses for the steel fish plates are to be the same as those used in Art. 22.

For the first design, let the depth of timber be again taken as 5 inches and let the thickness of the fish plates be provisionally

assumed as $\frac{3}{8}$ inch. The bending moment to be resisted by all the bolts in each timber is $32\,000\,(\frac{1}{2} \times 0.375 + \frac{1}{4} \times 5.0) = 46\,000$ pound-inches. If 8 bolts be assumed, the resisting moment for each bolt is 5750 pound-inches, thus requiring a diameter of $1\frac{3}{8}$ inches (see handbook). This, however, is not a standard size. The next larger commercial size has a finished diameter of $1\frac{7}{16}$ inches. The finished size must be used on account of its bearing in a steel fish plate. The compressive area furnished is $8 \times 5 \times 1.538 = 5.75$ square inches, thus indicating that the number of bolts assumed is correct.

The net width of the timber cross-section is $35.6/5 = 7.12$ inches, and the gross width is $7.12 + 2 \times 1.438 = 10$ inches. The net tensile section required for each fish plate is $32\,000/15\,000 = 2.133$ square inches. The net width of the plate is $10 - 2\,(1.4688) = 7.0624$ inches, the diameter of the hole being $\frac{1}{32}$ inch larger than that of the bolt. The required thickness of the plate is $2.133/7.0624 = 0.302$ inch. The thickness to be adopted is therefore $\frac{5}{16}$ instead of $\frac{3}{8}$ inch. The revised bending moment in each bolt then becomes 5625 pound-inches, and hence the diameter remains unchanged. It is also found that sufficient bearing area is provided for the bolts against the steel plate.

The length of the shearing surfaces behind each bolt is $256/(2 \times 8 \times 5) = 3.2$ inches and of the splitting surfaces $42.7/(8 \times 5) = 1.067$ inches. The longitudinal spacing of the bolts between centers is $3.2 + 1.067 + 1.438 = 5.705$ or $5\frac{3}{4}$ inches, while the distance from center of bolt to end of timber is 5 inches.

Since that part of the tensile stress due to the bearing of one of the end bolts is distributed over a width of 1.766 inches on each side of the bolt, the metal beyond the bolt should be extended far enough to enable it to act as a beam. The bending moment diagram for this beam is similar to that of a bolt in a

plain fish-plate joint (see Fig. 24*b*) and hence the maximum bending moment, which occurs at the middle, is $2000(\frac{1}{2} \times 1.766 + \frac{1}{4} \times 1.460) = 2500$ pound-inches. Placing this equal to the resisting moment of a beam of rectangular cross-section, having a width of $\frac{5}{16}$ inch and an allowable unit-stress in the outer fiber of 15 000 pounds per square inch, the depth is found to be 1.789 inches. The distance from the center of bolt to end of fish plate is then $1.789 + \frac{1}{2} \times 1.438 = 2.477$ or $2\frac{1}{2}$ inches. The total length of each fish plate is $2(3 \times 5.75 + 5.0 + 2.5) = 49\frac{1}{2}$ inches.

For the next design, let a depth of 6 inches be taken. Assuming the thickness of fish plate to be $\frac{3}{8}$ inch, and 6 bolts the bending moment, on each bolt is $54\,000 / 6 = 9000$ pound-inches. This requires a diameter of $1\frac{9}{16}$ inches, and hence the next larger commercial size must be adopted, the finished diameter being $1\frac{11}{16}$ inches. The compressive area is $6 \times 6 \times 1.6875 = 60.75$ square inches. On account of the excess in the resisting moment of the bolts and of the compressive area, let a computation be made to find whether the number of bolts may be reduced to 5. The result shows that the resisting moment is sufficient, but the compressive area is too small, for 5 bolts. The gross width of the timber required is 9.308 or 10 inches, the distance center to center of bolts is $6\frac{1}{2}$ inches, and from center of bolt to end of timber is $5\frac{5}{8}$ inches. The net width of the plate is 6.5624 inches and the thickness required is 0.3521 or $\frac{3}{8}$ inch, which equals that assumed. The distance from the center of the bolt to end of fish plate is 3 inches, and the total length of fish plate is $43\frac{1}{4}$ inches.

The detail drawing for the first design, with all the required dimensions, is given in Fig. 25*b*. The bill of material is as follows:

2 timbers, 5″ × 10″ × 20′ 0″	166 ft. B.M.
2 steel plates, 10″ × $\frac{5}{16}$″ × 4′ $1\frac{1}{2}$″	87.6 lb.
16 lateral pins, $1\frac{7}{16}$″ diam., 6″ long, with nuts	68.0 lb.

The estimate of cost is:

166 ft. B.M. lumber @ 3¢	$4.98
88 lb. steel plates @ 3½¢	3.08
68 lb. lateral pins @ 10¢	6.80
3 hours' labor @ 35¢	1.05
Total cost	$15.91

The cost of the alternative design is similarly made and found to be $19.35. A comparison of the cheapest designs of the four types of fish-plate joint may now be made, as follows:

1. Tabled fish-plate joint with wooden plates,	$10.32
2. Tabled fish-plate joint with steel plates,	13.17
3. Plain fish-plate joint with wooden plates,	21.13
4. Plain fish-plate joint with steel plates,	15.91

These results show the great advantage of avoiding the large lateral pins, the cost of which is so large a percentage of the entire cost of the plain fish-plate joints. The effect of the wooden fish plates in increasing the bending moment in the pins, and consequently the cost, is especially noteworthy. If, however, the lateral pins are replaced by double extra hydraulic tubing as described in Art. 24, half-inch bolts passed through the tubes, and special cast washers used to hold the bolts in position and take bearing on the wooden fish plates outside of the tubes, the cost of the joint is reduced from $21.13 to $11.76; the cost of the tubing cut to required length being estimated at 6 cents per pound. A similar design for the joint with steel fish plates reduces the cost from $15.91 to $12.95, thus making it less than that for the corresponding tabled fish-plate joint. Where the clumsy appearance of a joint is not objectionable, the cost may be similarly reduced by substituting ordinary bolts for the lateral pins. The cost of the bolts may be estimated at about one-half of that for the pins.

In an article in Engineering News, vol. 61, page 539, May 20, 1909, ROBERT FLETCHER calls attention to an old form of joint which has been used in New England for splicing tension mem-

bers. He commends it for its simplicity, directness of action, and certainty in the computation of its working value. As shown in Fig. 26c it consists of a bar of wrought iron or steel acting as a beam on each side of the joint, the corresponding ends of both bars being connected by ordinary bolts. Since the ends of the fibers bear on plain surfaces which are perpendicular to them, less width is required for the bearing area of the wood; and as there is no tendency to split the timber, the shearing surfaces are shorter than for round-bolt beams. The tenon bar may be made wide enough to resist its bending moment or to keep its deflection within any assigned limit, so that the pressure of the fibers may be regarded as a uniformly distributed load. The thickness of the bar must provide sufficient strength outside of the bolt holes.

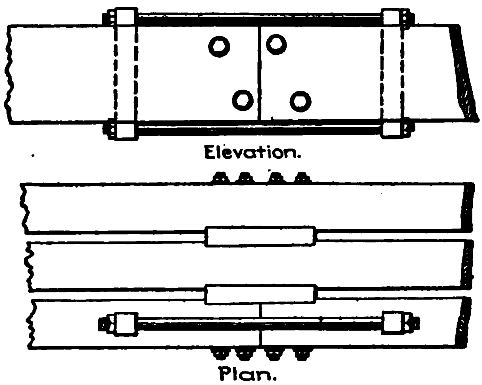


FIG. 26c. Tenon Bar Splice.

The splice is compact and short, since the shearing surfaces on both sides of the bar have a width equal to the larger dimension of the stick, with but small deductions for the horizontal bolt holes. It would be better if both of the transverse bolts near the end of the stick were placed directly below each other and as close to the joint surface as practicable. No difficulty is found in designing and constructing the joint to transmit fully 50 per-cent of the gross tensile strength of the stick. The carpenter work required is comparatively small and of such a nature that defects are not so liable to occur through careless workmanship as for other types of joint.

Fig. *26d* indicates by dotted lines the position of the holes cut to apply this joint to strengthen one of another type which was either not properly constructed or which has become weakened by partial decay. When used in splicing narrow timbers or planks the bars are connected by U-bolts, since their thickness is insufficient to allow for bolt holes through them. See Journal Association of Engineering Societies, vol. 15, page 21, July, 1895.

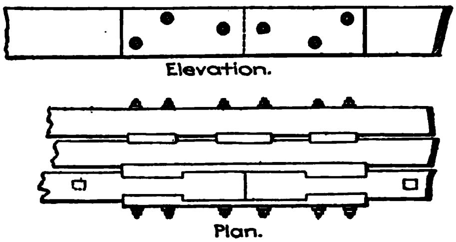


FIG. 26*d*. Lower Chord Splice.

Prob. 26. Determine the strength of the lower chord joint marked "Detail at A'' shown in Fig. 69*g*, assuming the same unit-stresses as those used in Arts. 22 and 26.

ART 27. LAP AND SCARF JOINTS.

In a lap joint one piece laps over the other at the connection. There are two general forms of the plain lap in use, one where the edges overlap as in a common form of weather boarding, and the other where the uncut ends overlap as in Fig. *27a* and are bolted together. Sometimes keys are inserted between the

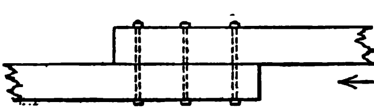


FIG. 27*a*. Lap Joint.

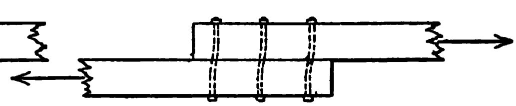


FIG. 27*b*. Lap Joint under Tension.

sticks to resist the longitudinal shear in the joint. This joint should not be used to transmit either tension or compression unless these stresses are small and merely incidental to the main function of the timbers spliced. The bending moment in the bolts is large in proportion to the longitudinal stress, and the tendency is to crush the fibers against the bolts near the joint surface as indicated in Fig. *27b*. The lap joint is not infrequently used to lengthen the boom or back stays of a

derrick, for which purpose the plain fish-plate joint is better adapted. See illustrations in Engineering Record, vol. 30, page 105, July 14, 1894.

A scarf joint is made by cutting away opposite sides from the ends of two timbers so that they may be lapped to form a continuous piece without increased thickness. There are a variety of scarfs depending upon the character of the cutting on each timber, some of which are ingeniously devised. In general the simpler forms are the best, since they can be more accurately fitted together and at less cost for labor.

The simplest scarf is that in which one-half of the thickness of each end is cut away as in Fig. 27*c*, and is therefore called the half lap. When used to splice compression members, its length is made about twice the depth of the timber and the bolts are placed at the quarter points. Its application to splicing

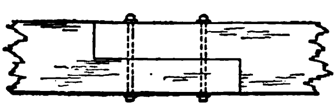


FIG. 27*c*. Half Lap.

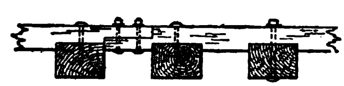


FIG. 27*d*. Splice in Guard Timber.

guard rails on bridges is illustrated in Fig. 27*d*, and to splicing the intermediate cap of a trestle bent, in Railroad Gazette, vol. 31, page 689, Oct. 6, 1899. In the plan of a wooden tower in Engineering Record, vol. 37, page 473, April 30, 1898, the half lap is combined with metal fish plates in splicing the columns. It is not economical to combine two types of joints in one splice.

Fig. 27*e* shows the addition of keys to a half-lap joint which is intended to resist some tension. This form has been used in

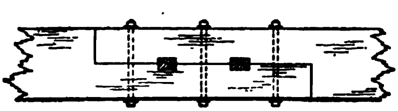


FIG. 27*e*. Half Lap with Keys.

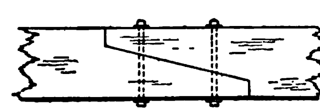


FIG. 27*f*. Oblique Scarf.

splicing the lower chords of wooden roof trusses in some exposition buildings, but it is not advisable to employ it when the

tension to be transmitted equals a large percentage of the net strength of the timbers to be spliced. It is also used in framing composite and laced posts (Art. 50).

The oblique scarf is shown in Fig. 27*f*. It is customary to cut down each end to one-fourth of the depth of the stick and to make the length of the joint from two to three times the depth. It will be noted that this form has more than double the flexural strength of the half lap. Fig. 27*g* indicates its

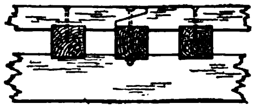


FIG. 27*g*. Splice in Guard Timber.

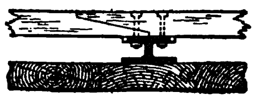


FIG. 27*h*. Splice in Running Board on Freight Car.

application to the splice of a guard rail and Fig. 27*h* to that of a running board on the top of a freight car. Fig. 27*o* shows the framing of a guyed pile driver in which the leads are spliced by oblique scarfs. It was built by JAMES H. FUERTES for use in driving trestle piles across very steep valleys on country roads. This form of scarf is also used in car sills other than the center sills to which the draw timbers are attached. When the inclined surface slopes in the opposite direction as in Fig. 27*i*, the joint is called beveled halving.

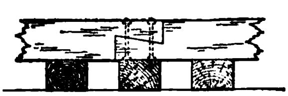


FIG. 27*i*. Beveled Halving.

The straight tabled scarf is illustrated in Fig. 27*j* and with the addition of a key in Fig. 27*k*. This joint can transmit some

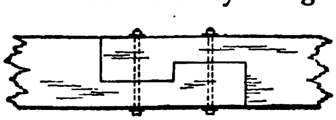


FIG. 27*j*. Straight Tabled Scarf.

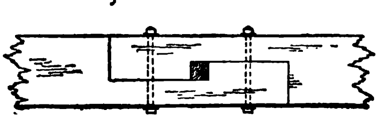


FIG. 27*k*. Tabled Scarf with Key.

tension as well as compression, and when the magnitude of the stress is known, the required dimensions of the parts can be readily computed. When the stress cannot be determined, the

length is made about $2\frac{1}{2}$ times the depth of the timber, while the height of the table is one-fourth of the total depth.

The oblique tabled scarf has two or more sloping surfaces of contact with an offset or shoulder between them, thus forming inclined tables. Fig. 27*l* shows such a joint with two tables on each timber. The height of the table equals that of the neck, which in this case equals one-fourth of the depth of the timber. The net section available for tension equals 50 percent of the

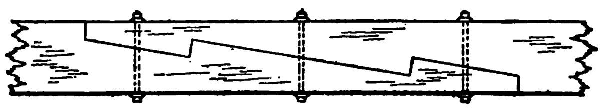


FIG. 27*l*. Oblique Tabled Scarf.

gross section of the stick. The eccentric application of the stress tends to develop considerable secondary bending in each part of the joint as well as in the splice as a whole. The great length required to secure sufficient shearing surface parallel to the fibers makes it an uneconomical form of joint. When there is but one table on each timber, the available net section is 33 percent. It will be observed that the bolts are not located in the sections containing the net depth as is the case with the straight tabled scarf.

Fig. 27*m* shows the same form with the addition of a key. In practice the depth of the table is often made about one-

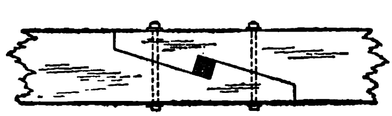


FIG. 27*m*. Oblique Tabled Scarf with Key.

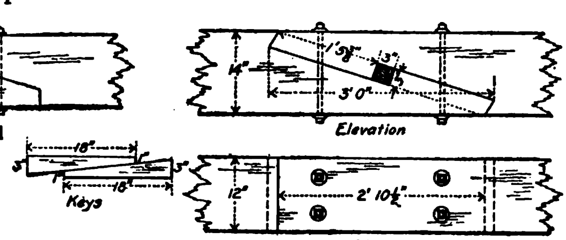


FIG. 27*n*. Splice in Sill of Framed Trestle.

fourth of the depth of the timber, while the length of the joint varies from $2\frac{1}{2}$ to $3\frac{1}{2}$ times the total depth. Fig. 27*n* gives the

form and dimensions of a splice in the sill of a high trestle bent copied from standard plans published in 1888. A simpler form of joint would probably be used at present. Other applications may be seen on the plans of a framed coffer dam in Engineering News, vol. 28, page 63, July 21, 1892, and of the crib for a waterworks intake in Engineering Record, vol. 35, page 534, May 22, 1897.

An important series of drop tests was made in 1908 by a Committee of the Master Car Builders' Association to determine the relative value of the half-lap and oblique-scarf joints in splicing the sills of freight cars. The impact was applied on the ends of spliced members 8 feet long, subjecting them to longitudinal compression, thus representing conditions which cause the largest number of failures of sills in service. The center or draft sills have a fish plate called a 'liner' added on one side of the splice to give it additional strength and stiffness. The sills were either of yellow pine or Douglas fir, while the liners were of oak or yellow pine. By comparing the average height of drop which caused failure for each type of joint, it was found that the combination of scarf and liner had a resistance of 23.5 percent of that of the straight sill, while the combination of the half-lap and liner had a corresponding resistance of 88 percent. The splices were also subjected to comparative transverse and tensile tests and in both cases proved the superiority of the half-lap over the scarf joint. For the detailed record of data and tests, including deflection and other diagrams, see Proceedings of the Master Car Builders' Association, 1909, vol. 43, page 000. An abstract of the report of the committee was published in Railroad Age Gazette, vol. 46, page 1489, June 24, 1909.

A committee of the same Association in 1902 had called attention to the relative weakness of various forms of the scarf as compared with the half-lap joint (called step splice by the

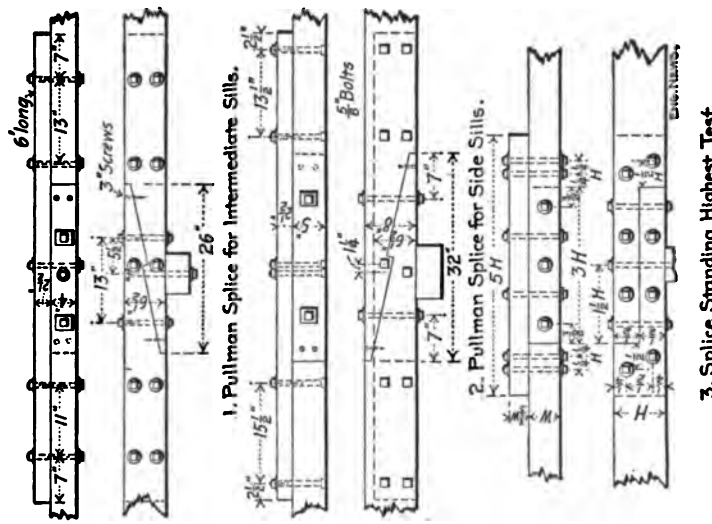


FIG. 279. Splices for Passenger Car Sills.

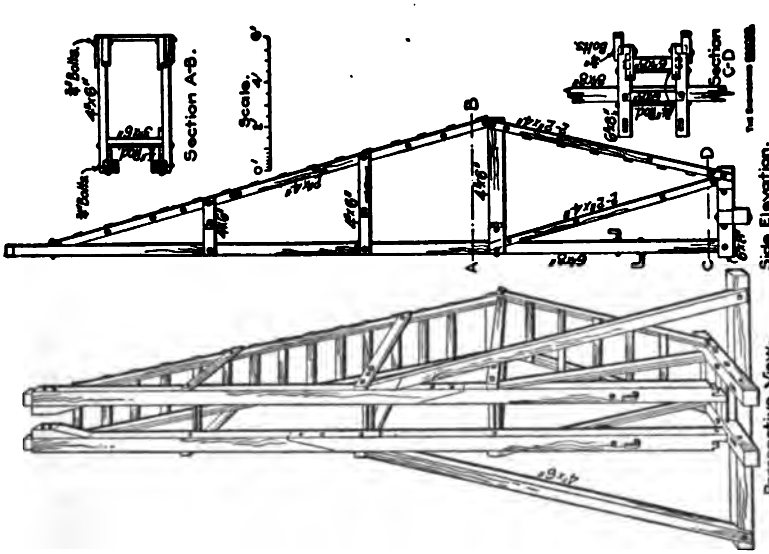


FIG. 270. A Guyed Pile Driver.

committee) in a report on the splicing of passenger car sills. The oblique tabled scarf without key is shown in Fig. 27*p* in the splices for intermediate and side sills; while the proportions of the joint recommended for adoption are given below them. A liner is also shown in each splice. The results of 22 compression and transverse tests on four types of splices made for the Committee are given in their report in Proceedings, vol. 36, page 137. The compression tests were without impact, and the methods of failure are shown by half-tone illustrations. The report shows all the kinds of car sill splices in use at that time.

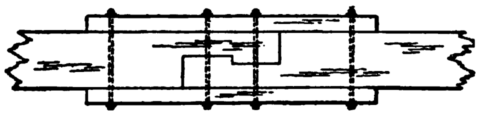


FIG. 27*q*. Splice in Sill of Trestle Bent.

An examination of framing plans published in engineering periodicals and in books on building construction reveals a large variety of designs for splices in which two different types of joints are combined, both intended to perform practically the same functions. Some of these combinations are as follows: Plain fish plate and half lap (Fig. 28*k*); plain fish plate and straight tabled scarf (Fig. 27*q*); plain fish plate and oblique scarf with keys (Fig. 27*r*); tabled fish plate on one side and plain fish plate on the other; plain and tabled fish-plate joint with steel plates (Fig. 69*g*).

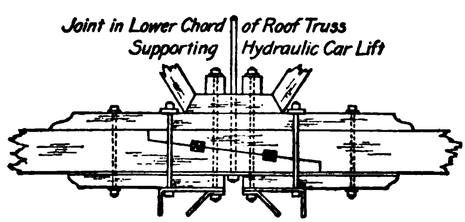


FIG. 27*r*. Composite Type of Joint.

Prob. 27. Refer to the Proceedings of the Master Car Builders' Association, 1902, vol. 36, page 137, and copy the dimensioned diagrams showing the proportions of the oblique scarf, oblique tabled scarf, and half-lap joints in car sill splices.

ART. 28. VARIOUS JOINTS IN BEARING.

A butt joint is one in which the end or edge of a timber comes squarely against another piece (Fig. 28*a*). This is the most effective joint between the ends of two compression members, the ends being held in place by other connecting braces as in Fig. 50*b*. In Fig. 28*b* the horizontal rails meet each other in

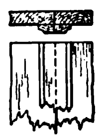


FIG. 28*a*. Butt Joint in Weather Boarding.

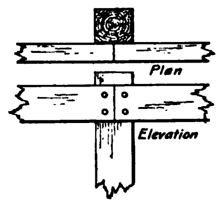


FIG. 28*b*. Butt Joint in Rail.

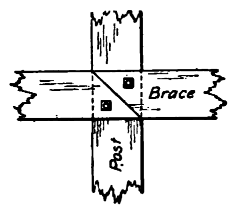


FIG. 28*c*. Bevel Joint in Brace.

a butt joint at their connection with a vertical post, but transmit little, if any, stress from one to the other. The use of iron straps and plates to prevent lateral displacement in the butt joint is described in Art. 18.

A bevel is any inclination of a cross-section other than a right angle. In Fig. 28*c* both braces are cut on a bevel, but in opposite directions.

A miter joint is made by beveling the pieces joined so that the plane of the joint bisects the angle between them, as at the corner of a picture frame. The application of this joint to tunnel timbering is shown in Figs. 28*d* and *e*. Unless the timber is well seasoned the transverse shrinkage of the wood causes the miter joint to open on the inner side and thus concentrate the compression on the outer edges. Another application is given in Fig. 28*f*, between the planks of which the rib is built up.

This joint is often used with some modifications at the hip of a truss as illustrated in Figs. 28*g* and *h*. The former is from a temporary truss for contractors' use, and the latter from a roof

FIG. 28*d*. Segmental timbering, used in tunnel work on the Western Pacific Railway. See Railroad Age Gazette, vol. 45, page 902, Sept. 11, 1908.

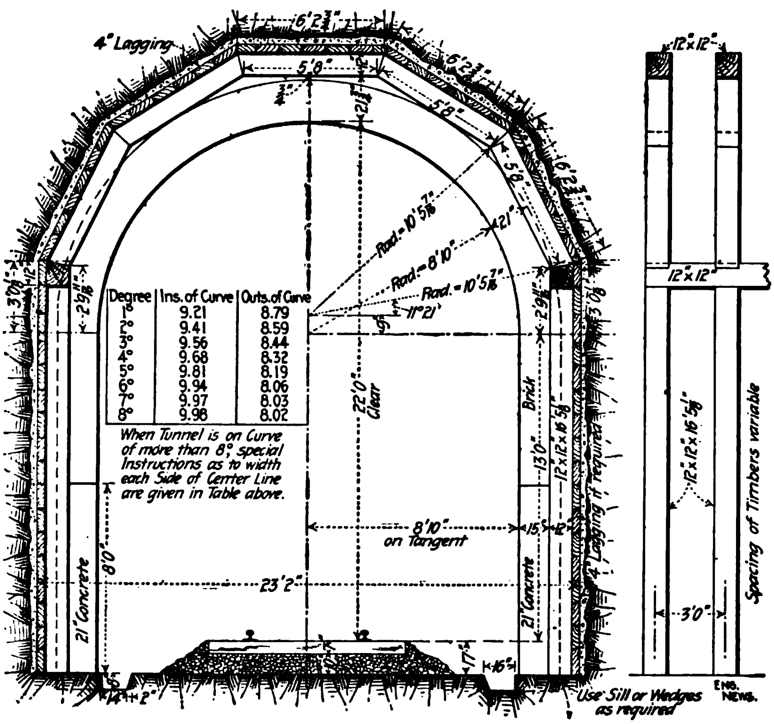
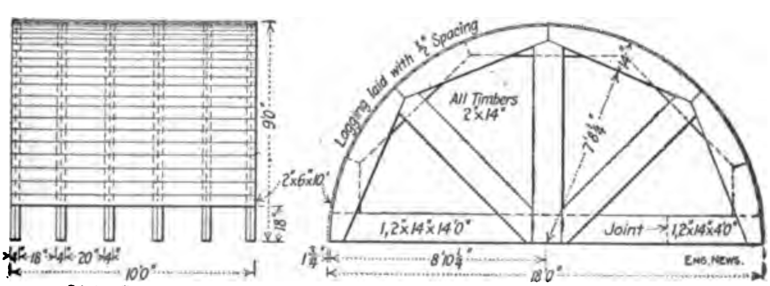


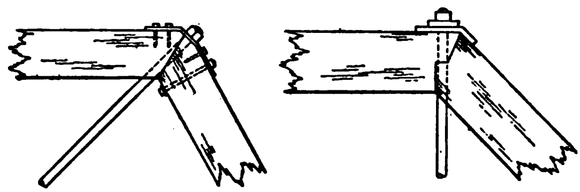
FIG. 28e. Timbering and Concrete Lining of the Langsville Tunnel on the Kanawha & Michigan Railway.



Side Elevation. End Elevation.

FIG. 28f. Arch Centering used in the Langsville Tunnel. See Engineering News, vol. 61, page 11, Jan. 7, 1909.

truss. In the latter case two washers are used in order to secure a larger bearing area without excessive thickness in the plate from which they are cut.



FIGS. 28*g* and *h*. Joints at Hips of Trusses.

Housing consists in letting the whole end of one timber for a slight distance into the side of another. The term 'housing' is also applied to the groove or recess cut into one piece into which

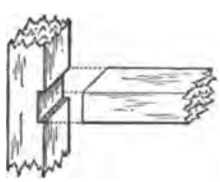


FIG. 28*i*. Housing.

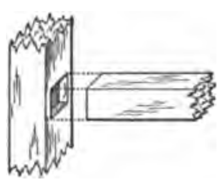


FIG. 28*j*. Housing.

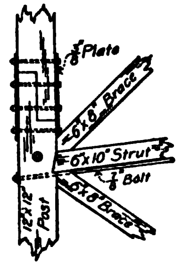


FIG. 28*k*. Intermediate Joint in Tower.

the other piece is said to be housed (Figs. 28*i* and *j*). In Fig. 28*k* the horizontal strut is let into a triangular groove in the post, and may be called beveled housing.

Notching consists in cutting a groove in the side of one timber to insert the side of another. When only one of the timbers is notched, it is called single notching (Fig. 28*l*); and when both timbers are notched, it is called double notching (Figs. 28*m*, *n*, *o*, and *p*). The term 'gaining' is often used instead of notching, especially in car construction. It is said that the timbers are 'boxed out' in order to 'gain' them into each other. In trestle construction the notching is often called 'dapping.'

No form of joint is probably used so extensively as notching in all kinds of timber structures, but especially in the framing of

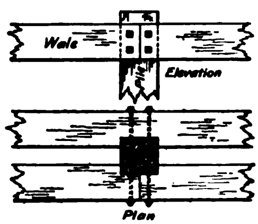
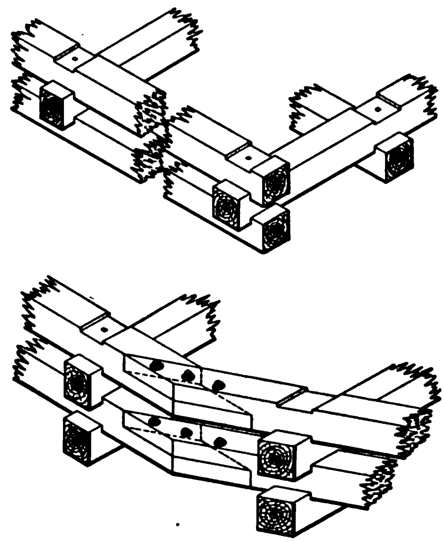


FIG. 28*l*. Single Notching.



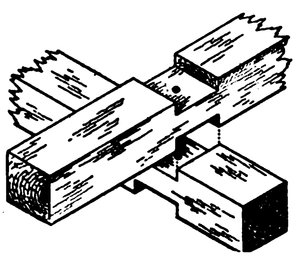


FIG. 28*m*. Double Notching.

FIGS. 28*n* and *o*. Details of Crib Framing.

cribs for foundations, docks, breakwaters, etc. Figs. 28*n* and *o* show the details of ballasted cribs used in New York City in municipal dock construction. See Engineering Record, vol. 58, page 525, Nov. 7, 1908. The economic value and efficiency of notching in the construction of a frame to support heavy loads was perhaps never so

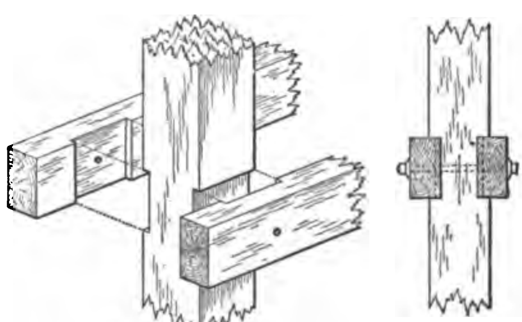


FIG. 28*p*. Double Notching.

well shown as in the falsework described in Engineering News, vol. 29, page 223, Mar. 9, 1893. Fig. 28*q*, which is reproduced

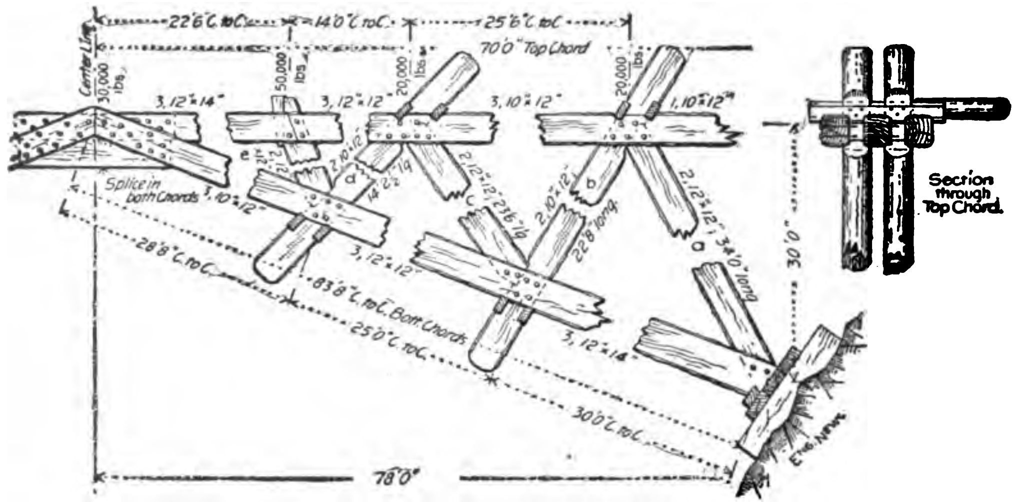
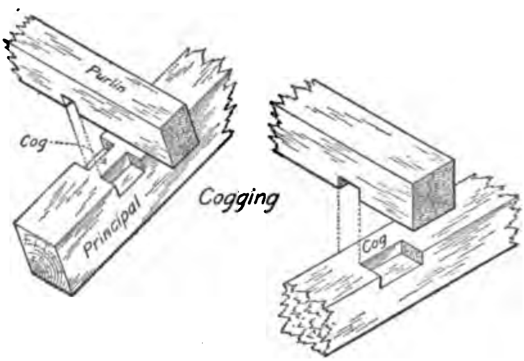


FIG. 28q. Half Arch of Falsework showing Details of Framing.

This novel trussed arch falsework was used to erect a steel bridge over Pend d'Oreille river at Albeni Falls on the Great Northern Railway, under unusually difficult circumstances. When the falsework was built in March, 1892, the snow was three feet deep, and the nearest railway point was 35 miles away, by trail through the woods. No sawed timber could be obtained except a few planks floated down the river. On account of the dangerous current the falsework had to be built out from the shore, making the connections in the air. Spruce and tamarack were selected for the compression members and a very tough fir for the tension members.

from that article, indicates such an arrangement of the truss that only two tension members were required in each half arch, thus simplifying the framing of the joints. The deck timbers were arranged to bring the loads only to the panel points. The only iron used in the falsework consisted of boat spikes $\frac{1}{2}$ inch square and 12 inches long. The article referred to describes further details and the difficulties encountered in the work owing to the location, and gives additional illustrations.

Cogging differs from notching in not having the cut on one of the timbers extending entirely across, as indicated in Figs. 28*r* and *s*. This kind of joint is used much less than formerly, when metal fastenings were not so abundant and cheap.



FIGS. 28*r* and *s*. Cogging.

In halving each timber is cut to one-half of its depth, so that when the two are united the upper and lower surfaces are flush.

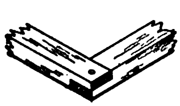


FIG. 28*t*. Halving.

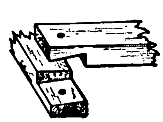


FIG. 28*u*. Beveled Halving.

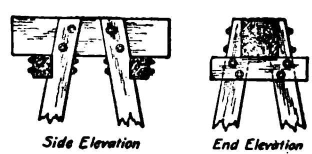


FIG. 28*v*. Bird's-mouth Joints.

When the pieces are joined lengthwise, it is a form of scarfing known as the half lap, but when the pieces cross each other it is a form of notching. Halving is a common method of joining wall plates or other timbers at a corner as in Fig. 28*t*. In beveled halving the surface of contact is inclined (Fig. 28*u*).

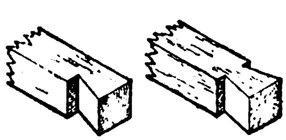
The bird's-mouth joint consists of an angular notch cut in one piece of timber so as to make it fit obliquely on the side of another. Both forms of this joint are shown in Fig. 28*v*, one on the side elevation and the other on the end elevation of the top of a cableway tower. The most frequent application of this joint is between a rafter and its supporting wall plate. Another application is shown in Fig. 28*d*, at the joint between the lower sticks of the timber arch and the plumb-post caps.

Prob. 28. Locate the bird's-mouth joint in Plate V.

ART. 29. DOVETAIL JOINTS.

A dovetail is a projection having the form of a truncated wedge on the end of one timber, intended to fit into a recess, or groove, of corresponding shape, in the side of another timber, so that the former may not be withdrawn in the direction of its length. When the dovetail is formed by cutting a triangular groove, or notch, only on one side of the timber, it is called a single dovetail (Fig. 29*a*), and when formed by cutting a notch on both sides, it is called a double dovetail (Fig. 29*b*).

This form of joint is used frequently in cribs for holding broken stone or concrete, and in cofferdams, since the pieces can resist both tension and compression. An example of its application may be seen in the cribs of a ship canal in



FIGS. 29*a* and *b*.
Single and Double Dovetail.

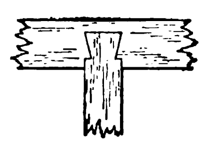


FIG. 29*c*.
Housed Dovetail.

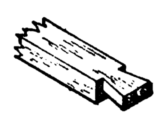


FIG. 29*d*.
Dovetail Halving.

Engineering News, vol. 40, page 50 (inset), July 28, 1898. In Fig. 29*e* the sides of the dovetails are sloped both ways, so that the dovetails on both sides of the corner of crib are alike. The housed dovetail is illustrated in Fig. 29*c*, and dovetail halving in Fig. 29*d*. The use of the last form, in an intake crib for a

waterworks, may be seen in Transactions American Society of Civil Engineers, 1895, vol. 34, page 28.

In 1876 a set of 36 tests was made by C. P. GILBERT, under the direction of General O. M. POE, Corps of Engineers, U.S.A., to determine the relative holding power of different forms of dovetails used in locking crib walls together by timber ties. The timbers were of undressed white pine, and 6 inches square.

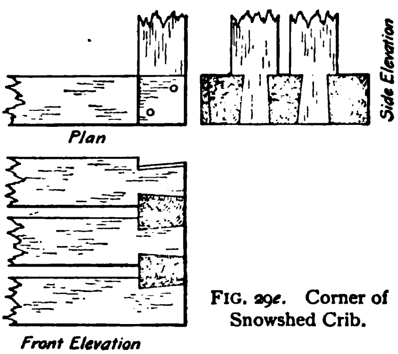


FIG. 29e. Corner of Snowshed Crib.

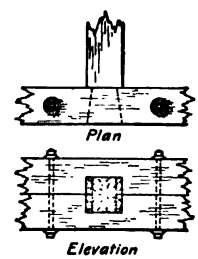


FIG. 29f. Joint for test.

The dovetail tie was secured between two pieces of the same section in notches cut out to fit the dovetail as indicated in Fig. 29f. The pieces representing the wall were united by bolts placed 6 inches from each side of the tie. The tension in the tie was resisted by reactions on the timber walls 15 inches on each side of the center.

The following conclusions were drawn from the tests: In the strongest form of double dovetail the depth of each notch equals one-sixth of the width of the stick. In the strongest form of single dovetail the depth of notch equals one-third of the width of the stick. The average strength of the double dovetails is greater than that of the single. The strongest form of double dovetail loses less from looseness at the point than at the neck. Smoothing the back of a single dovetail increases its strength. The position of the bolts does not materially affect the strength.

The average strength of the strongest form of double dovetail was found to be 12 915 pounds, and that of the single dovetail was 11 875 pounds, the difference being about 9 percent. Failure generally occurred in the walls, and only in a few cases did the dovetail shear. The results were originally published in Report of Chief Engineers, U.S.A., 1884, page 2069, and abstracted in Engineering News, vol. 24, page 549, Dec. 20, 1890.

Prob. 29. Compute the ultimate shear per square inch for the average total resistance given in the Engineering News for the ten forms and proportions of dovetail joints which were tested.

ART. 30. MORTISE-AND-TENON JOINTS.

In a joint of this type a tenon or rectangular projection on the end of one timber is inserted into a socket or mortise in the side of another stick (Fig. 30*a*). The length of the tenon is made

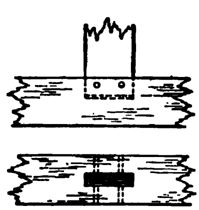


FIG. 30*a*. Mortise and Tenon.

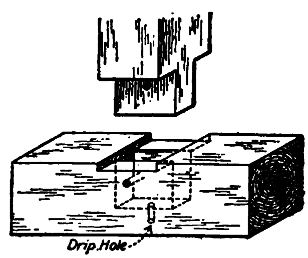


FIG. 30*b*. Housed Tenon.

a little shorter than the depth of the mortise so that the shoulders only may bear upon the sides of the other timber ; otherwise, the shrinkage of the timber containing the mortise may throw the entire weight on the tenon. The tenon is usually fastened in the mortise by either one or two wooden pins or treenails. The pins are placed at one-third of the length of the tenon from the shoulder. For timbers 10 to 12 inches thick the treenails should be from 1 to $1\frac{1}{2}$ inches in diameter with a taper of $\frac{1}{8}$ to $\frac{1}{4}$ inch. The hole in the tenon should be slightly nearer the shoulder than in the mortise, so that when the pin is driven in place it will draw the two timbers into close contact.

When the mortise-and-tenon joint is combined with housing, it is called the housed tenon, as illustrated in Figs. 30*b* and *c*. The housing gives additional bearing surface to resist the lateral displacement of the timber having the tenon. A drip hole is provided to drain the mortise of water that enters the joint

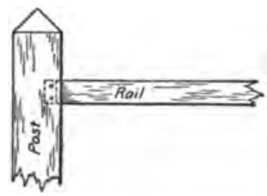
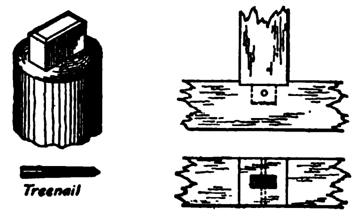


FIG. 30*c*. Housed Tenon.



FIGS. 30*d* and *e*. Stub Tenons.

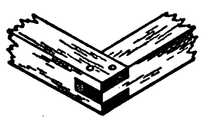


FIG. 30*f*. Open Mortise.

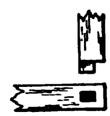


FIG. 30*g*. Eccentric Stub Tenon.

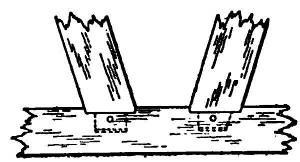
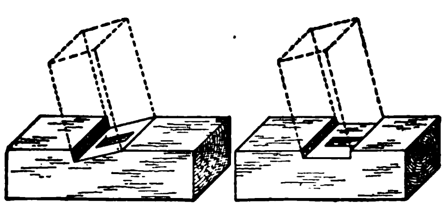


FIG. 30*h*. Tenons on Batter Posts.

when exposed to the weather. The stub tenon is illustrated in Figs. 30*d* and *e*, the former being at the head of a pile. When the joint is at the ends of both timbers, an eccentric stub tenon is sometimes used as in Fig. 30*g*, while in other cases an open mortise is employed as in Fig. 30*f*. See also the corner of a car



FIGS. 30*i* and *j*. Sills of Trestles.

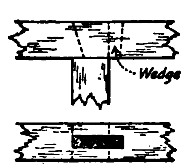


FIG. 30*k*. Dovetail Tenon.

underframe in Fig. 18*g*. Fig. 30*h* shows two forms of housed stub tenons when an inclined post meets a horizontal sill as in a wooden trestle bent. Figs. 30*i* and *j* show two forms of housing

for the housed tenon; in the first one the bearing surface for the shoulders is perpendicular to the pressure of the post, and in the second one the horizontal thrust of the post is resisted by the housing.

The dovetail tenon is illustrated in Fig. 30*k*. One side of the tenon is cut back into the form of a single dovetail, the mortise with receding sides is made wide enough to admit the broad end of the tenon, and then the tenon is held in place by a wedge driven along its back.

The tusk-and-tenon joint shown in Fig. 37*b* is practically obsolete. The tenon passes through the other beam and is held in place by a key. The tusk below the tenon is intended to form the principal bearing at the support. The double tenon is shown in the same illustration.

The double tenon as well as the tusk-and-tenon joints were formerly used in framing joists around a stairway, as in the joint between a tail beam and a header, or between a header and a trimmer. A tenon should not be used as a cantilever to carry a definite load. Its principal use is to prevent lateral displacement for a compression member in which an accidental blow may develop some flexure, or it may serve incidentally to resist tension while helping to maintain stiffness in a frame. Various forms of hangers have been devised in recent years to support one beam from another and thus to replace the joints mentioned (Art. 40). The application of the double tenon in the framing of lock gates is illustrated on the inset of Engineering News, vol. 33, page 98, Feb. 14, 1895, and in car underframes in vol. 35, page 111, Feb. 13, 1896; see also Fig. 74*d*.

In discussing the different methods of framing the bents of wooden trestle bridges, a Committee of the American Railway Superintendents of Bridges and Buildings described the advantages and disadvantages of the mortise-and-tenon joint as follows: "The mortise-and-tenon joint seems to be the one in

most general use, and if it were not for the extraordinary cost arising from the use of high-priced carpenter work, and the necessity for great accuracy in framing, it would be the best method for holding the timbers together. There are, however, several serious objections to this method, among which is the liability to rot on account of the mortise offering a receptacle for water, even though drained in a most approved manner. Every bridge man knows from his own experience and observation that the joints, especially where mortise and tenon are employed, decay long before signs of decay are visible at other points. This decay makes the renewal of the timber necessary, and as the balance of the stick is sound, much good timber is lost. This material can be, and generally is, used for shorter work, but at a cost nearly equal to new timber. Notwithstanding these defects in the mortise-and-tenon joint, it is the form generally in use and will probably continue to be as long as wooden bridge trestles are used. The advantages of its use are a maximum of strength and rigidity of the structure, which is the most important feature in the construction of a good trestle.

Prob. 30. Read C. D. JAMESON'S discussion on the mortise-and-tenon joint in Railroad and Engineering Journal, vol. 4, page 21, January, 1890. How does he finally characterize it ?

ART. 31. STEP JOINTS.

Stepping consists of one or more notches with inclined sides or bearings arranged in the form of steps. Two forms of the single step are shown in Figs. 31*a* and *b* respectively, the bottom of the notch being horizontal in the former. This joint is used very extensively in all kinds of timber structures. See storage bin at coaling station in Engineering News, vol. 28, page 436, Nov. 10, 1892; frame for supporting head gates and rack frame of a water-power plant in Engineering News, vol. 39, page 235 (inset), April 14, 1898; superstructure of temporary cribs of

waterworks intake tunnel in Engineering News, vol. 40, page 82 (inset), Aug. 11, 1898; and Howe truss used in underpinning

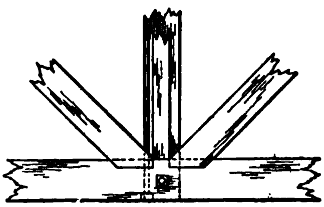


FIG. 31*a*. Single Step Joint in Car Underframe.

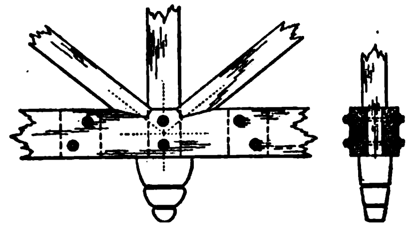
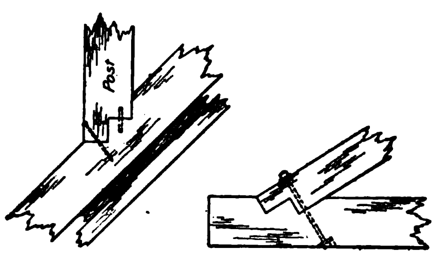


FIG. 31*b*. Single Step Joint at Middle of Roof Truss.

heavy walls in Engineering Record, vol. 31, page 25, Dec. 8, 1894. See also the upper ends of the braces in Fig. 69*b*, and the lower ends of two diagonal struts in Fig. 76*a*.

Double steps are illustrated in a joint of an ore dock frame (Fig. 31*c*); in the end of a queen-post truss (Fig. 31*d*); and in a railroad bumping block (Fig 31*e*). Another example is shown in the end joint of a roof truss in Fig. 31*f*. The same drawing



FIGS. 31*c* and *d*. Double Steps.

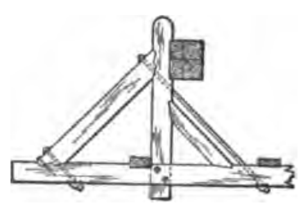
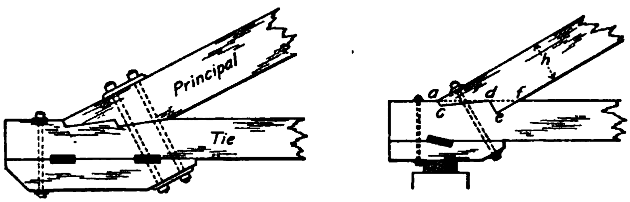


FIG. 31*e*. Bumping Block.

from which this was copied also shows the use of three steps, although it is questionable whether more than two can be advantageously employed in such a joint. See Engineering News, vol. 36, page 34 (inset), July 16, 1896. The objection to the preceding forms of double steps is the small shearing surface provided to resist the pressure on the second step. Fig. 31*g* shows how this defect is overcome. In this case a much longer shearing surface is secured by making the depth *de* larger than

ac, thus placing the shearing surfaces for the two steps at different elevations. In any case it requires very accurate workmanship to obtain the proper distribution of pressure on the two



FIGS. 31*f* and *g*. End Joints of Roof Trusses.

steps, but in Fig. 31*g* the eccentricity of the resultant pressure on the inclined compression member is materially less than in Fig 31*f*.

The application of single, double, and triple steps in a single structure is made in a sectional drydock, illustrated in Engineering News, vol. 45, page 314 (inset), May 2, 1901. See also plans of temporary wooden Pratt trusses in Engineering Record, vol. 35, page 269, Feb. 27, 1897.

An example of multiple stepping may be seen at the end joints of the bowstring trusses of the Ferry Bridges of the New York Passenger Terminal, Central Railroad of New Jersey, Engineering Record, vol. 59, page 650, May 22, 1909. See also Fig. 74*d*.

When the mortise-and-tenon joint is combined with a step, as in Fig. 31*h*, it is called the oblique tenon. These joints are

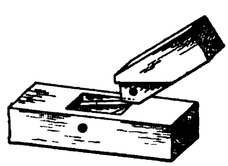


FIG. 31*h*. Oblique Tenon.

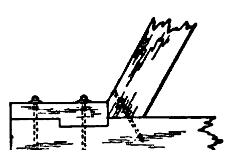
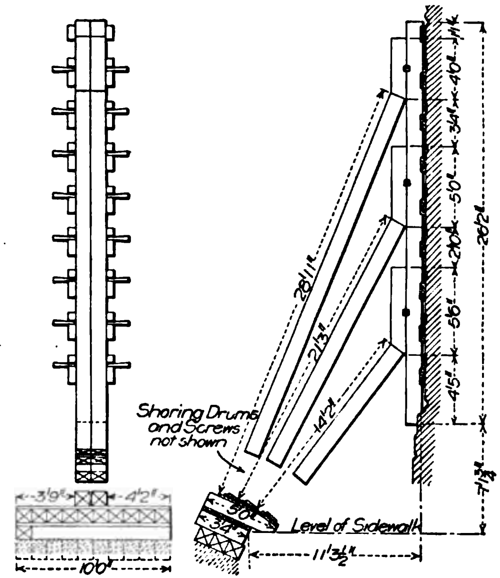


FIG. 31*i*. Head Block.

sometimes used in connecting struts to the chords of roof trusses but with the step omitted, so that the component of the stress



FIGS. 31*j* and *k*. Showing Ashlar Face Wall of the Farmer's Loan & Trust Company's Building in New York City

Oak blocks were fitted into the rectangular arrises of the cut-stone blocks. Corresponding horizontal oak pieces were keyed into the pair of vertical timbers. Pairs of folding wedges were then placed between them, and the load transferred to the inclined pushers and jackscrews. See Engineering Record, vol. 58, page 480, Oct. 31, 1908.

must be taken by the end of the tenon. Whether this is allowable in good practice depends upon the magnitude of the stress in the brace and whether the tenon provides sufficient bearing area for the specified unit-stress.

In temporary construction, as for example, in falsework, a head block is bolted to one timber to resist the longitudinal thrust of the other, thus replacing the steps. In Fig. 31*i* the head block has one table, for which keys are sometimes substituted. Usually, however, the head block is perfectly plain, and depends to some extent upon frictional resistance between it and the timber to which it is bolted. Another example of the use of head blocks with keys is illustrated in Figs. 31*j* and *k*.

While it is not customary to consider the friction of the timbers in designing joints with steps it may be desirable to do so in special cases. The friction must be considered when plain head blocks are used, and it will resist a considerable part of the thrust on any block when the latter is not plain, provided the bolts are drawn up to their full working capacity.

Prob. 31. Examine the framing of the Russell snowplow as illustrated in Railroad Gazette, vol. 23, page 743, and prepare a list of the different kinds of joints and fastenings used in its construction. Notice the timber knee and the manner in which it is employed.

ART. 32. ANGLE BLOCKS.

On Plate III are given the detail drawings for four cast-iron angle blocks for a wooden roof truss, the use of which simplifies the framing of the timber braces. In every case each end of a brace is sawed off square as for a butt joint, and a hole is bored to engage the protecting pin on the angle block, thus holding the timber in place. The smallest one is a solid casting, while the rest are hollow with a uniform thickness of metal throughout.

Fig. 32*a* shows another pattern of an angle block similar to No. 1 mentioned in the preceding paragraph. In this case both

ends are open. Holes are located in the middle of each side into which dowels may enter. These dowels are driven into the ends of the braces to hold them in place. The radial webs materially strengthen the block, which is intended for Howe trusses of spans less than 100 feet. For larger spans the angle blocks are more elaborate in form and contain tubes passing through the chord, so that the stresses in the vertical rods are transmitted directly through the angle block to the braces, thus avoiding the compression of the chord timbers on the side of their fibers. The details of separate rectangular tubes, as well as of two forms of angle blocks with open ends for a short span Howe truss bridge on an electric railway, are given on the inset accompanying an article in Engineering News, vol. 35, page 27, Jan. 9, 1896.

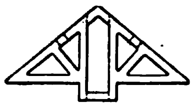


FIG. 32a. Angle Block.

The dimensioned detail drawings of a number of angle blocks and other castings for a temporary wooden roof truss used for exposition purposes may be found in Engineering News, vol. 28, 35, page 605 (inset), Dec. 29, 1892. Some of them are more complicated, since the type of truss adopted is not as well adapted to construction in timber as to construction in steel throughout. The same inset plate shows a cast-iron plate with short projecting pins or nipples on one side. A pair of such plates are used to connect a timber member to metal rods by means of a much smaller pin than can be employed if the bearing of the pin is directly on the wood. The projecting pins distribute the stress from the plate over a number of cylindrical bearing surfaces on each side of the timber. Angle blocks of wood are shown in illustrations contained in Arts. 69, 70, and 76.

Prob. 32. Consult Chap. IV in Keep's Cast Iron—a Record of Original Research, and ascertain the effect of different shapes in molds on the crystallization of cast iron. State the principles which must be observed in the design of castings to avoid planes or surfaces of weakness.

ART. 33. METAL SHOES.

The details of an end joint of a king-post truss shown in Fig. 33*a* has one of the most effective forms of shoe now in use for the inclined upper chord when the stress is too large to be resisted by double steps of the form given in Fig. 31*g*. The shoe is forged from a steel plate to fit the end of the chord, the right end being bent down to form a lug which engages a notch in the lower chord. The original illustration from which this is taken was published in Engineering News, vol. 21, page 306, April 6, 1889.

In designing the shoe, provision must be made for sufficient bearing at the toe and at the lug, while the metal must have sufficient thickness to resist the bending moment due to the lug

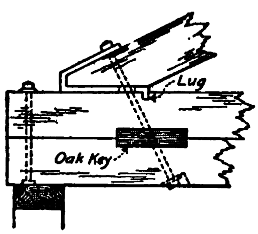


FIG. 33*a*. Forged Steel Shoe.

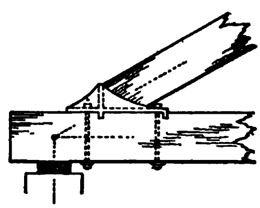


FIG. 33*b*. Cast-iron Shoe.

acting as a cantilever. The pressure at the toe and against the lug equals the horizontal component of the stress in the upper chord. The horizontal plate is subject to tension, but the unit-stress is very much less than that at its right end, which is due to flexure.

The shrinkage of the timber will release the tension put in the bolt when it is first drawn up, and after that the shoe has to move some distance horizontally before the bolt is again stressed in tension. Before that occurs, however, it will be subject to flexure on account of the pressure of the timber on the side of the bolt. These considerations indicate that the bolt and shoe cannot act together in resisting the horizontal thrust of the upper

chord. The sole function of the bolt is therefore to keep the members in place. The hole should be a little larger than the bolt, and as the pressure of the upper end of the shoe increases the stiffness of the fibers of the timber, the bolt should be drawn up to its approximate safe tensile strength.

A forged steel shoe can be designed to resist effectively the thrust at the end joint of the largest wooden roof truss that it is practicable to build. The plate may be extended, and one or more ribs riveted to it, as shown in Fig. 69*b*, or an extra bent plate may be added as on Plates I and II. A thinner plate may be used when the lug is omitted, and only ribs are riveted to the plate. It is very essential that the lug or end rib be held down either by the pressure of the timber above it or by a hook bolt through the lower chord.

Fig. 33*b* shows a cast-iron shoe having two lugs to engage notches in the lower chord. The horizontal thrust is received by a transverse web which is connected to both side plates and to the horizontal one. The vertical pressure of the shoe on the lower chord is larger at the left end than at the right, since the center line of the upper chord intersects the horizontal resultant reactions against the lugs on the left of the middle. Another form of cast-iron shoe is given in Fig. 33*c*. As indicated on the diagram the lower chord consists of two timbers separated one inch which equals the thickness of the vertical triangular web of the shoe that lies between them and which engages the inner side of each timber by its projecting lugs. Another lug engages the top of each timber. The transverse web plate which receives the thrust of the upper chord is stiffened by a small triangular web.

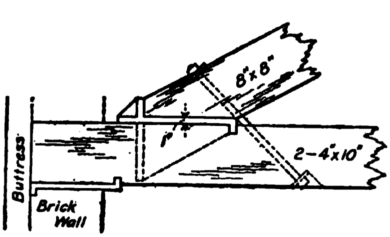


FIG. 33*c*. Cast-iron Shoe.

ART. 34. PRESERVATION OF JOINTS.

The joints of a wooden structure which is exposed to the weather are always the first to show incipient decay. The moisture enters between the surfaces of contact and remains after other parts of the timber become dry. It especially weakens the fibers at the ends of any timber by saturating the cell walls of the wood at the interior as well as at the surface of the stick. For this reason the diagonals of a Howe truss fail by crushing at their bearing on the angle blocks. Alternate wetting and drying causes rapid decay known as wet rot.

To increase the durability of a structure it is therefore important to treat the contact surfaces of timber at the joints with some preservative material. For this purpose creosote oil, common tar, and asphaltum, when applied very hot, are valuable and comparatively cheap. White lead and linseed oil are used to some extent, but are more expensive. Bolt holes or other holes bored in any part should be thoroughly saturated with the preservative, while all bolts, including drift bolts, should be warmed and coated with the same material. Coal tar may also be used for bolts and holes, no heating being necessary.

When the conditions are such as to make it economical to treat the timbers throughout, the framing should be done before subjecting the timbers to the preservative process. If any additional cutting is required after treatment, the exposed surface should be given several coats of the preservative with the brush, sufficient time being allowed for each coat to be absorbed by the wood before the next one is applied.

Various patented preparations, sold under the names of avenarius carbolineum, fernoline, woodiline, etc., are used to a considerable extent for the preservation of wood, all of them being either applied with the brush or by open tank treatment. To obtain the best results the materials are applied hot.

Wood preservation is made the subject of comprehensive study and investigation by one of the standing committees of the American Railway Engineering and Maintenance of Way Association. See the first report in vol. 10 of the Proceedings (1909) giving statistics, considering preservatives, with specifications, adaptability of woods for treatment, the strength of treated timber, and treating processes.

A marked difference in the effect of oak and yellow pine wood upon iron bolts was discovered upon examining an old wooden bridge which was removed in 1902. Rods $1\frac{1}{4}$ inches in diameter were found to be reduced by corrosion to a diameter of about $\frac{1}{2}$ inch where they passed through oak blocks; while there was no perceptible corrosion where they passed through the yellow pine timbers.

CHAPTER III.

WOODEN BEAMS AND COLUMNS.

ART. 35. DESIGN OF WOODEN BEAMS.

Wooden beams are nearly always rectangular in section. When they are designed for strength only, the width and depth must be determined so that the unit-stress in the outer fiber as well as the horizontal shear at the neutral surface do not exceed their respective safe values. When only the stress in the outer fiber is considered, it is shown by mechanics that $bd^2 = 6M/S$; in which b and d are the width and depth of the beam respectively, M is the maximum bending moment, and S the unit stress in the outer fiber. Since the strength varies as the square of the depth d, the value of d should be assumed first, and then that of b found from the value of bd^2.

When only the unit shearing stress is considered, $bd = 3V/2S_h$ in which V is the maximum vertical shear and S_h the unit stress in horizontal shear. If the same security is desired in regard to both of these stresses, the value of d is found on dividing the former by the latter equation given above, whence $d = 4S_hM/SV$. For a simple beam having a span l and a load W uniformly distributed, its value is found to be $d = lS_h/S$, or $d/l = S_h/S$. These equations show that the depth of the beam is directly proportional to the span, and that the ratio of depth to span is the same as the ratio of the allowable unit stresses at the neutral surface and in the outer fibers respectively. For structural timbers this ratio ranges from one-eighth to one-fourteenth. When, however, the entire load is concentrated at the middle, the ratio becomes $d/l = 2S_h/S$. Similar relations

may be determined for any beam or condition of loading by using the values of the maximum bending moments and vertical shears, which may be found in the manufacturers' handbooks or in any text-book on mechanics.

For smaller depths than those given in the preceding paragraph, the strength of the beam is governed by the unit-stress in the outer fiber, while for larger depths the strength depends upon the horizontal shear.

When the conditions under which beams are employed require them to be designed for stiffness, the dimensions of the cross-section must be found, so as to keep the deflection within a given ratio to the span. Such conditions apply to joists or beams which support a plastered ceiling in a building, or a line of shafting in a machine shop or factory; or to stringers in wooden railroad bridges.

The deflection is usually limited from 1/360 to 1/480 times the span for ceilings, and to 1/200 of the span for the stringers in railroad bridges and trestles. In mill construction, where the deflection of floors must be limited on account of supporting lines of shafting, the allowable deflection, according to C. J. H. WOODBURY, is that which causes the beam to bend into a curve with an average radius of 1250 feet.

In designing the beam by means of the formula for deflection, it is important to remember that the modulus of elasticity for a load applied continually is only about one-half as large as for a load which is removed within a few minutes after its application. The dead load is, therefore, to be doubled and added to the live load, the sum being substituted for the load W in the deflection formula, provided the live load is also distributed with at least approximate uniformity. Otherwise, it is necessary to solve the problem by trial. In this case the approximate size of the beam may be found by considering the live load only, afterwards computing the deflections for the dead and live loads sepa-

rately by means of the respective formulas, and then comparing their sum with the allowable deflection. In some cases it should also be considered whether a beam is liable to receive its maximum load when still unseasoned, or only after it has been allowed to season.

Since the stiffness of a beam of rectangular section varies as the cube of the depth and directly as the width, the least material will be required when the section is just sufficient to keep the unit-stress in horizontal shear within the safe limit. It is assumed that the unit-stress in the outer fiber is usually also safe when this condition is fulfilled. When, however, the depth exceeds 12 inches, the least material may not give the least cost, for the cost of dimension timber is a function of the depth or section area, as well as of the volume expressed in feet B.M.

If C_1 and C_2 denote the costs per foot B.M. for two beams of depths d_1 and d_2 respectively, it may be readily shown that the total costs of the beams are to each other as $c_1 d_2$ is to $c_2 d_1$, provided the beams are of equal strength depending upon the stress in the outer fiber, the span and loading being the same. If c_3 and d_3 are the corresponding cost and depth of a third beam having the same deflection as the first one, their total costs are to each other as $c_1 d_3^2$ is to $c_3 d_1^2$. It will be found, accordingly, that for depths less than a certain value the beams must be designed by the formula for strength, and for greater depths by the deflection formula. As previously noted, the beams must be safe in all cases with respect to horizontal shear.

It is shown by mechanics that the elastic resilience of a beam is a measure of its resistance to external work or of its capacity to absorb shock. The resilience of a beam increases with its section area, and if only this function of the beam be considered, it is immaterial whether the long or the short side of the section is placed vertical. When two beams have similar cross-sections, their resiliences are directly proportional to their volumes.

The most resilient beam of rectangular cross-section which can be cut from a circular log has its width equal to its depth (see Fig. 35*a*). The strongest beam of rectangular cross-section which can be cut from a circular log has the relation between its width and depth as 1 to $\sqrt{2}$, or nearly as 5 to 7. If D be the diameter of the log, the sides of the cross-section are $b = D\sqrt{\frac{1}{3}}$ and $d = D\sqrt{\frac{2}{3}}$. These values indicate a simple graphical

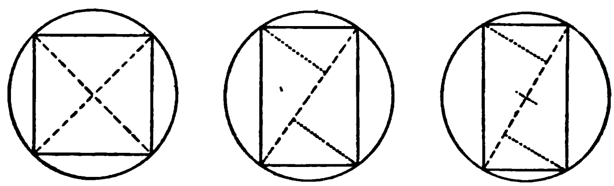


FIG. 35*a*. FIG. 35*b*. FIG. 35*c*.

construction of the cross-section as shown in Fig. 35*b*, by dividing the diameter into three equal parts, erecting perpendiculars, and uniting the extremities of the perpendiculars and diameter. The stiffest beam of rectangular cross-section which can be cut from a circular log has its width and depth related as 1 to $\sqrt{3}$, or as 1 to 1.732, or as 0.577 to 1. The width equals one-half of the diameter of the log and the graphical construction of the cross-section is indicated in Fig. 35*c*. It is interesting to note that the depths in the three cases are related as $\sqrt{1} : \sqrt{2} : \sqrt{3}$, the width in each case being unity.

When many beams have to be designed, it is convenient either to prepare tables for beams of various depths and spans and one inch wide, like those published in Cambria Steel, or like those of KIDWELL and MOORE originally published in Proceedings of Association American Railway Superintendents of Bridges and Buildings, 1908, page 205; or to construct diagrams for the purpose.

An ingenious diagram for proportioning wooden beams, either for strength or stiffness, was designed by CARL S. FOGH and

published in Engineering News, vol. 32, page 244, Sept. 27, 1894. A very simple diagram giving by inspection the dimensions of a wooden beam for a given span and load was constructed by J. M. MICHAELSON and is published in Transactions American Society of Civil Engineers, vol. 25, page 231, Aug., 1891. Both diagrams, however, do not consider horizontal shear, and hence it is necessary to revise the design in case the ratio of depth to span exceeds that indicated in a preceding paragraph.

Another convenient diagram was given in an article by F. E. GIESECKE in Architecture and Building, vol. 23, page 94, Aug. 24, 1895. It gives by inspection the sizes of yellow pine floor joists for dwellings spaced so as to give a deflection at the center equal to 1/400 of the span under a uniform load of 50 pounds per square foot. The ordinates represent the spacing in inches, and the abscissas the clear span in feet, the curves indicating the nominal dimensions of joists varying from 2 by 4 to 2 by 14 inches in section. The actual dimensions of the commercial sizes were, however, used in making the computations for the diagram.

Sometimes beams may be used in a manner which makes it impossible to determine the loads and reactions with any precision. An example of this kind is found in the railroad cross-tie. As an excellent illustration of the treatment of such problems in which theoretic investigation is applied to conditions assumed in accordance with experience and good judgment, the student is referred to A Study of the Stresses existing in Track Superstructure and Rational Design based thereon, by O. E. SELBY, in Proceedings of American Railway Engineering and Maintenance of Way Association, 1907, vol. 8, page 52.

Prob. 35. Find the uniformly distributed load which a 6 by 12-inch Norway pine beam with a span of 12 feet can carry for the specified stresses of 1000 pounds per square inch in the outer fibers and of 70 pounds per square inch at the neutral surface; also for a deflection of 1/200 of the span.

ART. 36. TESTS AND INSPECTION OF WOODEN BEAMS.

The importance of a careful inspection of the timber used for beams and of the location of knots and other defects is shown by numerous tests of large or full-size sticks. Of 209 spruce beams tested at the Massachusetts Institute of Technology, 142 failed on the tension side, 23 failed by horizontal shear only, and 10 failed on the compression side. Of the remainder in which the failure was a combination of two or three of these methods, 33 included tension, 15 included shear, and 24 included compression.

In another series of 1121 tests given in Circular 115 of the U. S. Forest Service (1907), 251, or 22.4 percent, of the beams failed by horizontal shear. Over two-thirds of the beams were Douglas fir, while the rest were Western hemlock, loblolly pine, longleaf pine, Norway pine, and tamarack.

In tests of 112 full-size bridge stringers made at the University of Illinois, 71, or 63.4 percent, failed by horizontal shear, 39 failed by tension, and only 2 by compression. An analysis was also made of the influence of knots and other defects on these failures, and it was found that of the beams which failed by horizontal shear, 9 were influenced by defects, while 23 of those which failed on the tension side were influenced by defects, the defects in both groups being chiefly knots.

With but few exceptions it is found that the ultimate unit-stress in horizontal shear is higher for the beams which fail by shear than for those which fail on the tension side. This fact, together with the comparatively few failures on the compression side, proves that the weakening effect of knots is most serious when they are located near the lower side of the beam.

A special series of 93 tests of green loblolly pine beams was made by the U. S. Forest Service to determine the weakening effect of knots. The sticks were 5 by 12 inches and had

a span of 15 feet; 34 were loaded at the center and the rest at the third points of the span. When a knot, wavy grain, or defect occurred in the lower quarter of the stick and within the middle half of its length, the stick was classed in group 1; sticks having defects in the corresponding upper quarter but not in the lower were classed in group 2; while those having defects elsewhere than in the parts named were put in group 3. Of the beams loaded at the third points the average strength for groups 1 and 2 was found to be about 75 and 80 percent respectively of that for group 3, while for those loaded at the center the corresponding percentages are about 79 and 100. The tests of small pieces cut from the large sticks indicated that a part of the difference in strength is due to the quality of the wood fiber. The sticks in group 3 naturally contained close and firm growth, since rapid growth and knots usually occur together. It is worthy of note that no failure of any beam in group 3 was influenced by a defect which determined its classification.

A similar series of 135 tests of Douglas fir beams loaded at the center gave an average strength for groups 1 and 2 equal to about 80 and 93 percent of the strength of group 3. In this case the portions of the stick to which the defects were referred extended over two-thirds instead of one-half of the span. See Second Progress Report on the Strength of Structural Timber, by W. KENDRICK HATT, Circular 115, U. S. Forest Service, 1907.

The principal reason why a knot near the tension side of a beam weakens it, is that the fibers which are deflected upward or downward in passing the knots are subject to tension across the grain, the resistance to which is very low in all timber. If a knot is on the compression side of a beam, it also weakens it, although not in the same degree as in the previous case, because the corresponding fibers are subject to compression across the

grain, which is much less than the compression parallel to the grain. In either case the tension or compression is accompanied by shear parallel to the curved grain.

These facts indicate why the following sentence is inserted in standard specifications for stringers of railroad heart grade; these stringers being usually about 16 inches in depth: "Knots greater than $1\frac{1}{2}$ inches in diameter will not be permitted at any section within 4 inches of the edge of the piece." The specifications are published in the Manual of the American Railway Engineering and Maintenance of Way Association.

For a record of some tests of full-size bridge stringers, in which is shown the effect of placing the defects on the upper or lower sides, and which confirms the practice of using the best edge in tension, see an article by ONWARD BATES on Pine Stringers and Floor Beams for Bridges, in Transactions American Society of Civil Engineers, vol. 23, page 261, November, 1890.

Season checks or ring shakes near the neutral surface materially reduce the resistance of a beam to horizontal shear. The former class of defects may be most effectively prevented by properly piling and covering the beams during the early part of the seasoning process. This recommendation is supported by comparative tests of groups of new and old Douglas fir stringers, the latter being eleven years in service. It was found that the percentage of old stringers which fail by horizontal shear is the same as that for new stringers, while the percentage of failures by knots is also the same. The average unit-stress developed in the two groups differs less than 3 percent.

Prob. 36. Refer to Forest Service Circular 115 of the U. S. Department of Agriculture, and study the comparative bending strength of large and small sticks, including the extreme fiber stress at the elastic limit, the modulus of rupture, and the modulus of elasticity.

ART. 37. FRAMING OF BEAMS.

To ascertain the effects of bevels, notches, and mortises on the strength of beams, a series of tests of twelve forms of small wooden beams was made by WILLIAM R. KING in 1881, the results of which were published in the Report of the Chief of Engineers, U.S.A., 1883, page 1496. The span, form of beam, and manner of failure are shown in Fig. 37*a*. The beams were made of Tennessee poplar, and the timber was selected so as to secure a uniform quality. The breaking weight, as given, is the average of two tests which generally differ less than 5 percent.

One of the most remarkable facts shown is that a beam may be greatly strengthened by cutting away certain portions so as to conform more nearly to a beam of uniform strength. Thus, beams 3, 5, and 8 when cut away to the form of beams 4, 6, and 7 become respectively 118, 134, and 32 percent stronger. The weakness of the forms, with shoulders near the supports, is mainly due to the great difference in stiffness on opposite sides of the shoulder and the resulting tendency to split, as indicated in the diagrams.

In 1893, CHARLES BABCOCK made a similar set of tests at Cornell University with beams of the same clear span, but 1 by 2 inches in section, and cut down at the ends to a depth of 1 inch. Five different kinds of wood were used, viz. white pine, yellow pine, chestnut, ash, and oak; and the increase in strength due to cutting away the shoulders, so as to leave the full depth for a length of 4¼ inches, ranged from 13 to 207 percent. The method of failure in each case was due to horizontal shear, or rather a combination of shear with tension across the grain.

On comparing the ultimate loads for beams 1, 3, and 8 in Fig. 37*a*, it is seen that a shoulder of only 20 percent of the depth reduces the strength 36 percent, while a shoulder of 60 percent of the depth reduces it 73 percent.

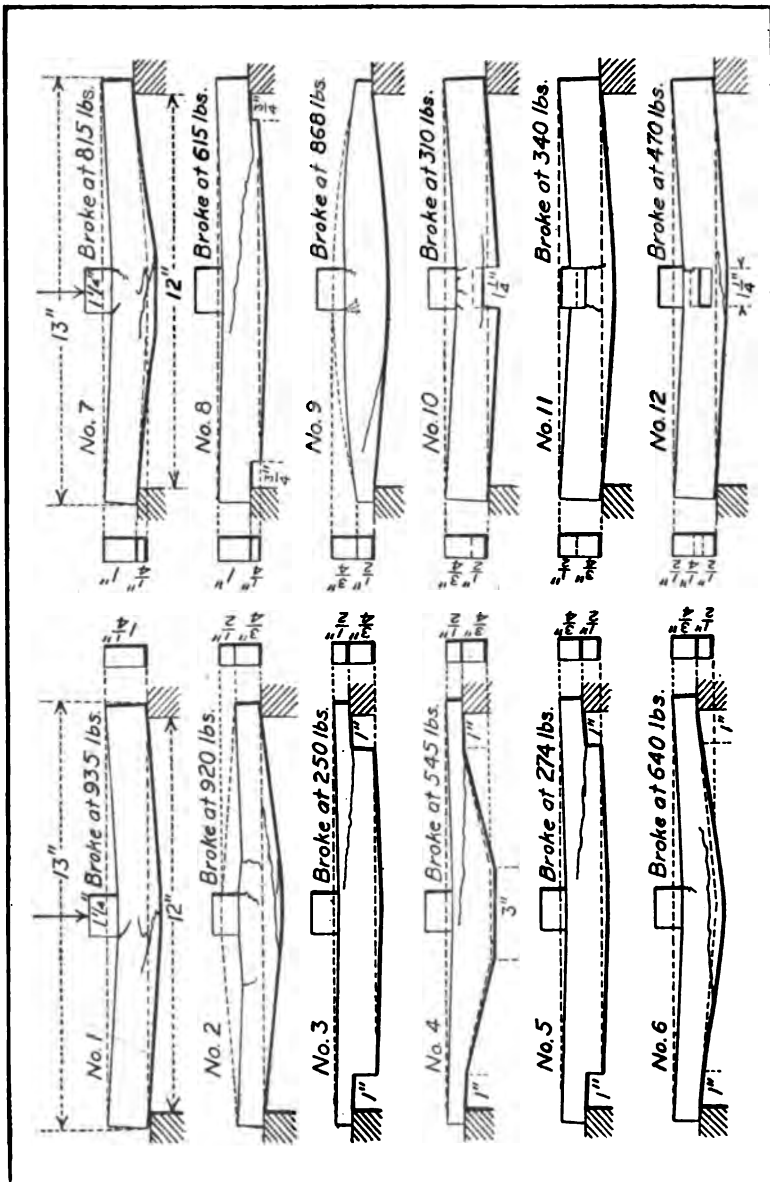


FIG. 37*a*. Tests showing the Effect of Notches, Mortises, etc., on the Strength of Beams.

For BABCOCK's tests the percentage of reduction in strength due to a shoulder of one-half of the depth of the beam is as follows: white pine, 71; yellow pine, 66; chestnut, 70; ash, 70; and oak, 51. These facts emphasize the practical waste of material due to the common practice of cutting such shoulders on joists in buildings, and when conditions rarely justify it.

The notching of cross-ties over stringers when the spacing of the stringers materially exceeds that of the track rails, or of long cross-ties over deck trusses, leads to similar results, and this fact has been observed by bridge inspectors. The following extract is from an article by JUSTIN BURNS on the Inspection of Timber Structures (see Engineering Record, vol. 38, page 181, July 30, 1898): "If the Howe truss is a deck bridge, the track will probably be carried on ties resting on the top chords of the trusses. These ties are usually 14 or 16 inches in depth, their size, of course, being dependent upon the distance between trusses. They are notched down a few inches over the chord to prevent their lateral movement and these notches should be observed for cracks, or the splitting sway of the lower portion of the tie. Theoretically, the notched tie is as strong a beam as one of the same depth but unnotched, yet this square notch has a weakening effect, as the timber tends to split off below the notch."

Exception may be taken to this reference to theory, since theoretically as well as practically any radical change in cross-section in a member, whether it is subject to tension or to flexure, deflects the lines of stress from their normal direction and thereby weakens it. The loss of strength is not due only to cutting away material, but also to the development of secondary stresses.

The breaking loads for beams 10, 11, and 12 show the weakening effect of a notch or mortise at the middle of a beam, but the beam is not so reduced in strength as when a shoulder is made at the ends by cutting down the depth.

At a stairway or other opening through a floor, the beams on the sides of the opening are called trimmers; the cross beam which is near the head of a person passing up or down the stairway is called the header, and the joists or beams supported by the header are known as tail beams.

A series of tests was made at the Massachusetts Institute of Technology to find the strength of timber headers loaded through tail beams framed into the headers by tusk-and-tenon joints. As the test was to be of the headers and not of the tail beams, the latter were made quite short. In some cases each

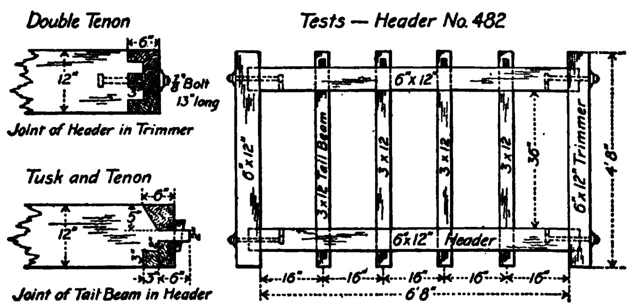


FIG. 37*b*. Arrangement of Beams for Tests.

end of the header was framed to the trimmer by a double tenon joint and held in place by a joint bolt, while in others the header was supported by a stirrup and held in place by a joint bolt. The dimensions of the joints and the general arrangement are shown in Fig. 37*b*, the number of tail beams ranging from 4 to 7.

All the headers gave way by splitting along the line of mortises cut to receive the tusks and tenons of the tail beams. In every case where the headers were framed into the trimmers by double tenon joints the end below the lower tenon also split off.

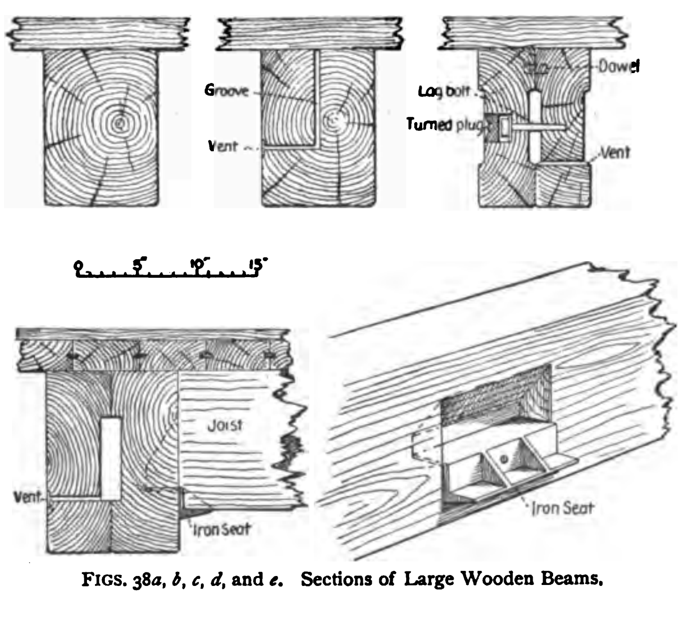
The general results show a reduction of strength in the hard pine headers varying from 50 to nearly 90 percent, due to the cutting involved in the method of framing. With a material where the shearing strength along the grain is relatively so

small as in timber, any cutting whatever injures the strength and stiffness of the beam. It is therefore much better to avoid framing whenever possible and to use stirrups or some approved form of hangers, both for the tail beams and headers. The detailed record of the tests and half-tone illustrations showing the methods of failure were first published in Technology Quarterly, vol. 10, September, 1897, and afterwards reprinted in pamphlet form.

Prob. 37. Compute the ultimate value of the maximum horizontal shear in beam No. 1 in Fig. 37*a* and its probable corresponding value in beam No. 3.

ART. 38. CONSTRUCTION OF WOODEN BEAMS.

Fig. 38*a* shows the section of a large beam in which season checks extend outward from the heart since the heart dries out



FIGS. 38*a*, *b*, *c*, *d*, and *e*. Sections of Large Wooden Beams.

more slowly than the outer wood of more recent growth. Fig. 38*b* shows a method of admitting air to the interior by means of

a saw cut extending through a part of the depth and providing vent holes at intervals. Another method which is more frequently adopted is to cut the stick vertically into halves and to place the heart sides outward as well as reversing one piece. This arrangement affords the additional advantage of inspecting the heart, which in mature trees begins first to decay.

Fig. 38*c* shows the two halves with panels planed on the sides, thereby admitting air also to the interior. The vent holes are placed at the bottom of the central groove and can therefore drain off any water that may collect. They are joined by lag screws which do not become loosened so much by the shrinkage of the wood as if bolts are used extending clear through. The dowels in the top help to equalize the deflection, and therefore also the loading in cases like that shown in Fig. 38*d*. The panels on the outside materially improve the general appearance and reduce the strength but slightly. See Engineering News, vol. 28, page 78, July 28, 1892.

While the methods described in the preceding two paragraphs permit better inspection and prevent serious checking in beams as the timbers dry out in the structure, there is an additional advantage in building up a beam or girder of two or more sticks placed side by side, since the smaller sticks cost less than the larger ones, and are more readily obtainable in the market. Such a beam is sometimes called a compound or composite beam, and the extreme case is one in which a series of planks are fastened together side by side with spikes or bolts. If every plank extends the full length of span, the combination is fully equal and in most cases stronger than a solid beam of the same external dimensions, since with proper inspection large knots and other defects in the inner planks, which materially impair the strength, will cause their rejection, while in a solid stick only the outer surfaces can be thoroughly inspected. It is necessary, however, in case spikes are employed to fasten the planks

together, that some through bolts be provided to prevent the outer planks from buckling.

Fig. 38*f* shows the general plan of one span of a pile trestle with ballasted floor. The floor consists of creosoted timber and each stringer is made up of two pieces, 3 by 16 inches, to secure more thorough creosoting than can be done with a solid stick. The sticks are spiked together with 5-inch spikes spaced about 12 inches apart; each stick extends over two spans, and is bolted at its center to the cap of the bent. The arrangement of the stringers shown, avoids the necessity of cutting off the end of any stick which may be too long, since it is very objectionable to do any cutting or framing of timbers after they have been creosoted. It also has the advantage of allowing the floor to be readily strengthened by putting in additional stringers without disturbing the roadbed or existing stringers. This illustration was originally used in an article by R. MONTFORT, then Chief Engineer of the Louisville and Nashville Railroad, in Railroad Gazette, vol. 30, page 151, March 4, 1898.

The advantages of using two timbers instead of one in trestle construction are indicated in the report of a Committee of the American Railway Superintendents of Bridges and Buildings, in 1892, on Frame and Pile Trestles: "One method of obviating many of the defects in the mortise-and-tenon joints is by the use of double caps, posts, and sills. That is, in place of using 12 by 12-inch timbers, two timbers 6 by 12 inches can be used; and being properly fitted and securely bolted together, not only give good results as to strength and durability, but expose all defects frequently found in the center of large timbers. Also, being of only one-half the size and weight, they can be handled much more rapidly and consequently much more cheaply. The parts to be renewed are rendered more accessible, this being especially the case where trains are to be carried while the renewals are made. No other method of constructing trestles secures such

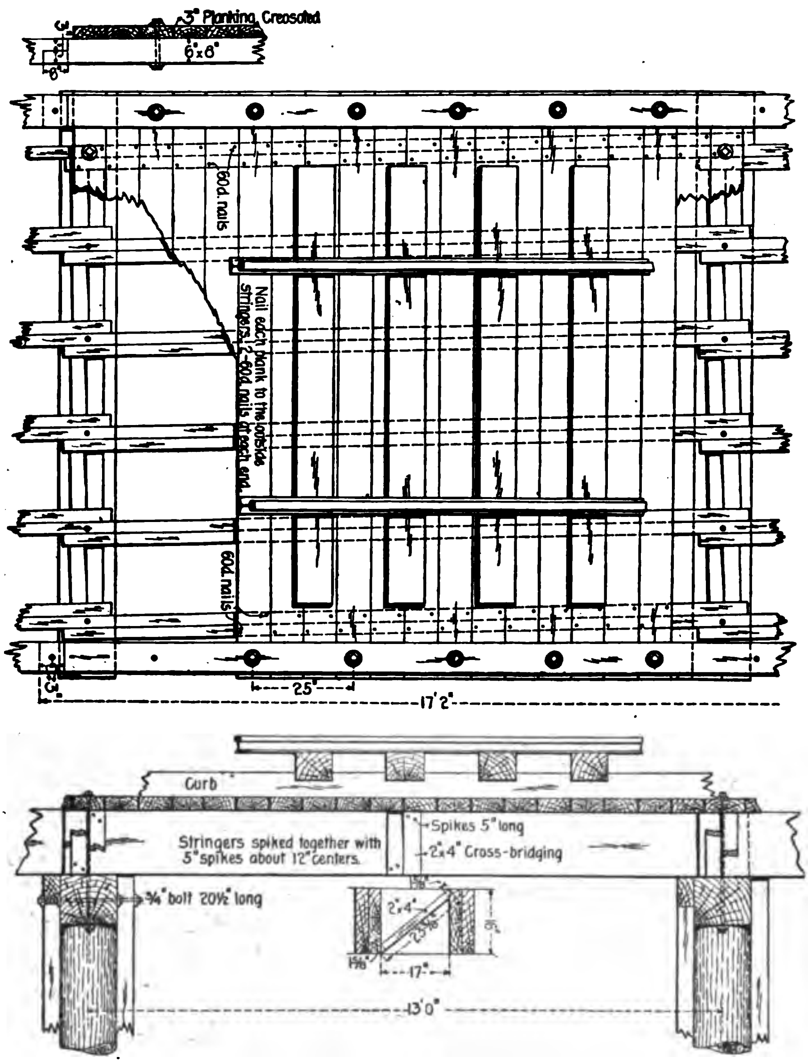


FIG. 38*f*. Plan and Longitudinal Section of One Span of Pile Trestle with Ballast Floor — L. & N. R.R.

great economy in renewals, and it has all the advantages of the ordinary mortise-and-tenon joint." One distinctive feature of this construction requires especial emphasis, namely, that any

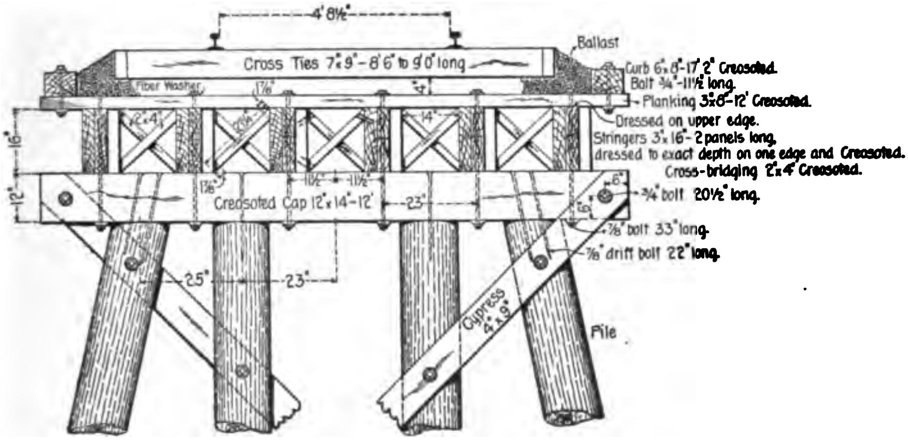


FIG. 38*g*. Cross-section of Pile Trestle with Ballast Floor.

one piece may be renewed without disturbing the rest of the structure. For a discussion on the strength of posts composed of more than one piece, see Art. 50.

On account of the fire risk involved with narrow open spaces between timbers in buildings it is considered better to bolt them together as tightly as possible, depending upon shrinkage and barometric changes to give all the necessary opportunity for access of air to the inner surfaces. See letter of JOHN R. FREEMAN in Engineering Record, vol. 55, page 194, Feb. 23, 1907.

Prob. 38. Compute the modulus of rupture, ultimate horizontal shear, and modulus of elasticity of the beam composed of three oak liners bolted together, the data for which are given at the bottom of the table in Railroad Age Gazette, vol. 46, page 1489, June 24, 1909. Compare these results with the corresponding unit-stresses given in the table in Art. 82.

ART. 39. PACKED STRINGERS.

A packed stringer is made up of a number of pieces of equal depth placed side by side but not in contact, and bolted together. Such stringers are commonly used in the ordinary type of wooden trestles with open floors. A stringer is placed directly beneath each rail. They vary in depth from 12 to 20 inches; while a depth of 16 inches is most frequently used, the modern heavy loading requires depths of 17 to 20 inches. For light loads two sticks may be used under each rail, but for the heavier loads three or four sticks are necessary. The span ranges from 12 to 16 feet.

As trestles are exposed to the weather, the sticks cannot be placed in contact, since moisture would collect between them and start decay. To avoid this they are spaced from one to two inches apart, although sometimes the clear distance is even

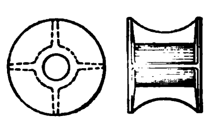


FIG. 39*a*. Packing Spool.

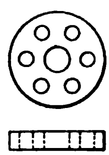


FIG. 39*b*. Separator.

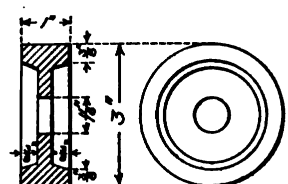


FIG. 39*c*. Separator.

greater. The spacing is secured by cast-iron separators or spools, or by wooden packing blocks. The iron devices are preferable as they are not so liable to gather moisture. Numerous forms of separators and spools are employed, Figs. 39*a*, *b*, and *c*, showing one form of spools and two forms of separators. A substitute for a separator consists of two washers with their larger flat sides against the timbers and the smaller ones in contact. Fig. 73*b* shows the plan of a steel packing plate which also serves as a splice plate to aid in holding together the ends of the stringers. The plate is separated from the timbers by thin cast-iron separators. A packing splice is also indicated

in Fig. 39*d*. When no bolsters are used, it extends below the stringers and is notched over the cap to hold the stringer in place. Four ¾-inch bolts are usually employed at each joint. Timber packing blocks are objectionable on account of holding moisture and promoting wet rot in the adjacent sticks.

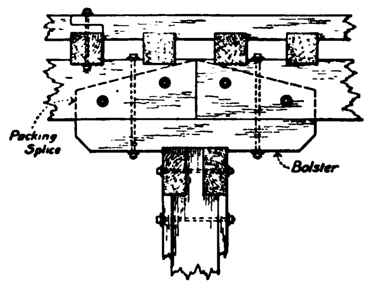


FIG. 39*d*. Packing of Stringers.

When it is possible to secure the timber of the requisite length, the sticks composing the stringers should extend over two spans and break joints over the caps of the bents. The arrangement of the bolts shown in Fig. 39*d* is such that when used with steel splice plates the stringers have been known to carry a train in safety when one supporting bent was washed out. The lower part of the splice plate is in tension in such an emergency, and hence the lower bolts are placed farther apart to give a larger resistance to shearing or splitting in the ends of the timbers.

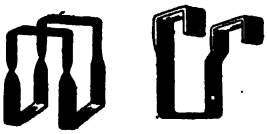
To secure the stringers against shifting laterally or longitudinally the best practice is to fasten the middle of each continuous piece to the cap by a drift bolt. The bolts should extend into the cap about two-thirds of its depth.

Prob. 39. Consult Foster's Wooden Trestle Bridges and prepare a table giving the spacing for the sticks composing the stringers as shown on the various standard plans.

ART. 40. BEAM HANGERS.

The first metallic hanger to be introduced as a substitute for the joints referred to in Art. 30, and which is still in use, is the stirrup or stirrup iron. The single form illustrated supports one end of a joist, while the double form supports two joists or beams on opposite sides of a beam or girder (see Figs. 40*a* and *b*). In determining the section of steel flat bars to be used, the most

important considerations are to provide sufficient bearing area on the side of the fibers for the joist or beam, and to make the bars of sufficient thickness to resist the bending moments at the center or edges of the lower bearing and at the edges of the upper bearings. If the bars are too thin, they will bend materially, and thus fail to distribute the bearing pressure uniformly.



FIGS. 40*a* and *b*. Double and Single Stirrup.

For example, let it be required to design a stirrup to support a load of 2100 pounds from a girder 6 inches wide. The spruce beam which bears on the stirrup is 4 inches wide. For an allowable compression on the side of the fiber the width of the stirrup bar must be $2100/(4 \times 270) = 1.945$ or 2 inches. The pressure of the stirrup on top of the girder may be assumed to vary from zero at the rear edge to its maximum value at the front edge. The load on each arm of the stirrup is $2100/2 = 1050$ pounds, and the lever arm of the reaction of the girder against the stirrup arm is $6/3 = 2$ inches, making the bending moment $1050 \times 2 = 2100$ pound-inches. The resisting moment of the stirrup section at an allowable unit-stress of 15 000 pounds per square inch is $15\,000 \times 2\,t^2/6 = 5000\,t^2$ pound-inches, t being the thickness of the bar in inches. Equating the two moments, t is found to be 0.6481 or 11/16 inch.

The average compression per square inch on the side of the fibers of the girder is $1050/(2 \times 6) = 87.5$ pounds and the maximum compression 175 pounds, which is below the specified limit. If it were assumed that the pressure at the front edge of the girder is 270 pounds per square inch, then the point of zero pressure would be less than 4 inches from that edge, and the bending moment thus produced would be less than that computed in the preceding paragraph. If each half of the stirrup bar under the end of the beam be regarded as a cantilever, the

bending moment in the bar at the edge of the beam is also found to be less than that previously computed.

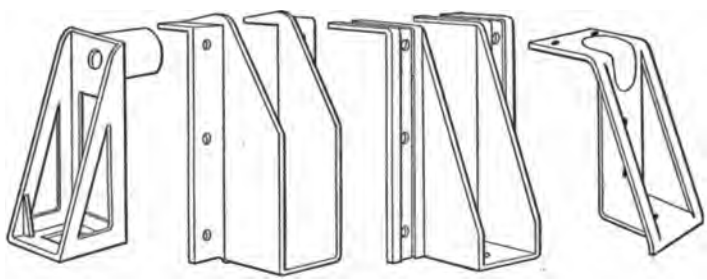
Stirrups are frequently used to support the purlins of a roof truss when conditions do not favor resting the purlins directly on top of the upper chord, as well as to support the joists of a ceiling from the lower chord of a truss. The use of a stirrup in suspending a beam below a window cap in a factory building is illustrated in Engineering Record, vol. 43, page 526, June 1, 1901. The same illustration shows a double stirrup, with lag screws inserted below to keep the beams from sliding off their seats.

A number of patented forms of beam hangers of wrought iron, malleable iron, or steel have been introduced, and are used in structures of the best design. Fig. 41*d* shows the Goetz joist hanger, which is made of wrought iron and was introduced in 1885. The illustration shows the hanger resting on cast-iron anchor plates, and when so arranged is called a wall hanger. The ends of the joist hanger are inserted into holes bored into the supporting beam or girder above the neutral axis, and as the ends are bent down the hanger tends to keep in position.

Fig. 40*c* shows the duplex hanger for a narrow beam, which is made of malleable iron; another form is for a joist meeting the supporting beam at an angle of 45 degrees, while the third form is adapted to large size timbers, being used in pairs. The cylindrical bearing of this hanger has a horizontal axis. The projecting lug at each side is intended to engage a notch in the joist. In computing the end bearing of the joist it will be noted that an allowance must be made for the opening in the bottom of the hanger. This type of hanger was first placed upon the market in 1895. In computing the safe load for the Goetz or the duplex hanger it may be assumed, according to the results of tests, that it is limited only by the safe bearing value

of the cylindrical bearing surfaces on the side of the fibers of the supporting beam. As shown in Art. 23, the effective bearing area equals the horizontal projection of the cylindrical surface when the direction of pressure is perpendicular to the fibers.

Figs. 40*d*, *e*, and *f* show three forms of pressed-steel joist hangers. They are known as the national, Van Dorn, and ideal



FIGS. 40*c*, *d*, *e*, *f*. Duplex, National, Van Dorn, and Ideal Beam Hangers.

steel joist hangers respectively. In each case the hanger is supported on top of the beam, but in Fig. 40*d* the resistance to bending at the upper corners is increased by extending the vertical plates above the adjacent bearing plates. The increased stiffness thus secured helps to distribute more uniformly the pressure on the top of the beam. In Fig. 40*f* the increased resistance to bending is obtained by sacrificing the bearing surface. Fig. 40*g* shows an additional form of hanger which resembles the stirrup in some respects. It aims to distribute the pressure on the supporting beam over a larger surface. The elements of weakness in hangers when not designed for the loads they have to carry have been demonstrated by building failures. See Engineering News, vol. 48, page 420, Nov. 20, 1902; and also vol. 49, pages 58 and 128, Jan. 15 and Feb. 5, 1903.

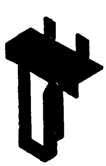
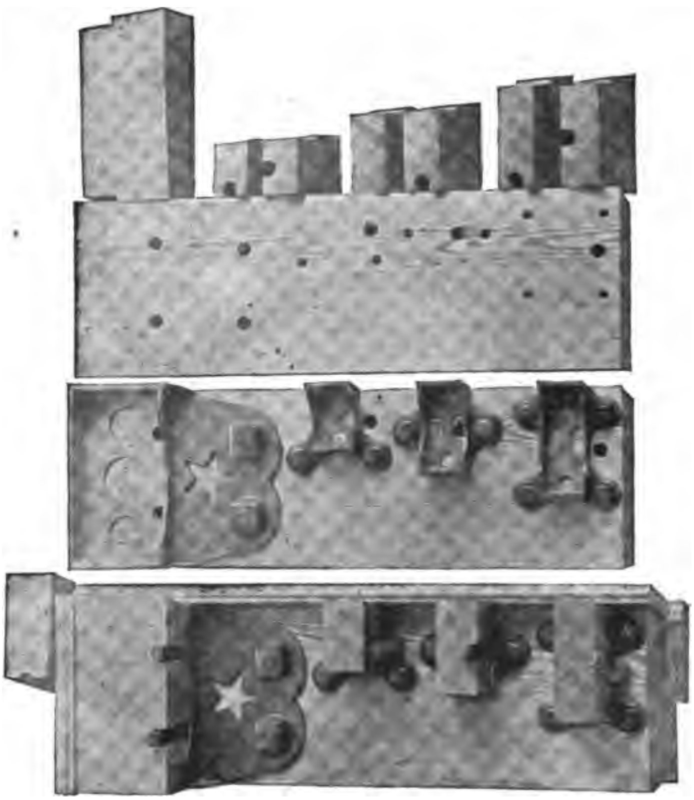


FIG. 40*g*.

The old method of framing carlines into plates in freight-car construction is indicated in Fig. 40*i*. The weakening of the plates by mortising is strikingly manifest. The end pieces of carlines of different sizes show that their strength is reduced



R. R. Gaz.

FIG. 40*h*. Carline and Sill Pockets.

by cutting the shoulders needed to form the tenons, in the same manner as for other beams as described in the preceding article. The malleable iron carline and sill pockets in Fig. 40*h* perform the same function for carlines as beam hangers do for ordinary joists or beams. The bottom view shows several pockets of various sizes attached to the plates with the plain ends of the

carlines in place; the middle view shows the same pockets with the carline and sill ends removed; and the top view indicates

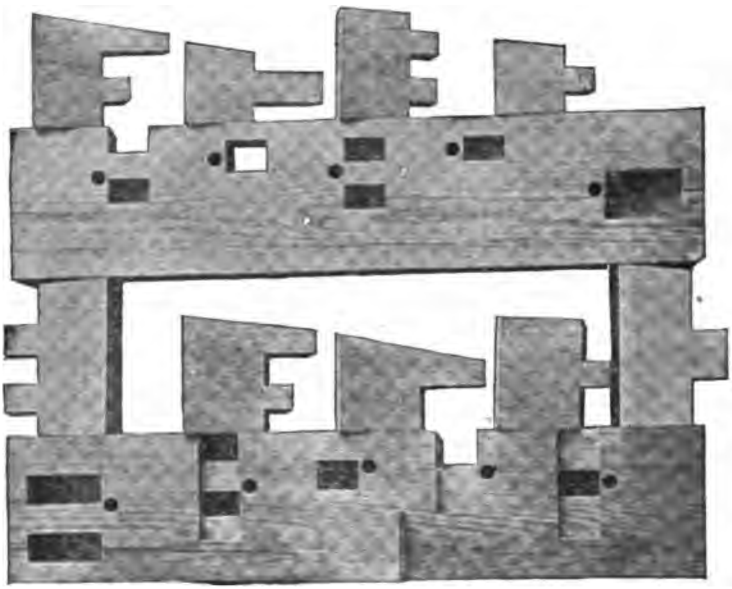


FIG. 40*i*. Portions of Carlines and Plates showing Weakening of Timbers by Mortising.

the small amount of material removed by drilling for the connecting bolts.

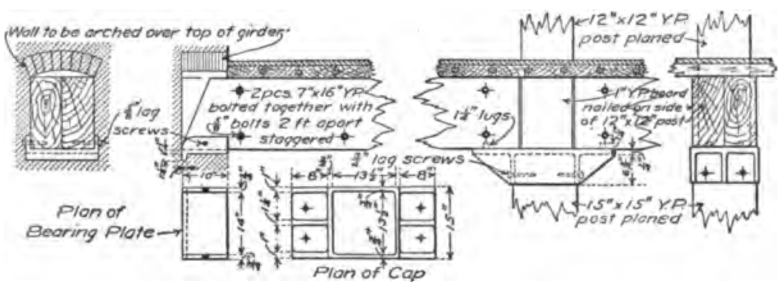
Prob. 40. Secure the catalogues issued by manufacturers of beam hangers and make a critical comparison of the results of tests given, and note the manner in which the load is applied in case a hanger is not tested in a regular testing machine.

ART 41. ANCHORAGE OF BEAMS.

A large number of devices have been used in practice to anchor a beam or joist to a wall which supports its end. Some of them are of such flimsy construction that they can be regarded as anchors in appearance only. Some of those which are effective in certain respects are dangerous in others, since they overturn the wall in case of fire when the beam gives way. In the

best construction the anchorage is arranged to release the beam in such an emergency without damage to the wall.

In Fig. 33*c* the beam is supported on a cast-iron wall plate which has a lug at the rear to hold the plate in position, and another lug inside of the wall to engage a notch in the bottom of the beam. Since the beam has no bearing on the right of the

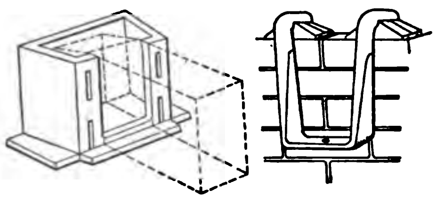


FIGS. 41*a* and *b*. Cast-iron Bearing Plate and Post Cap.

notch it is weakened by it (Art. 37). This defect may be remedied by locating the lug nearer to the end of the beam as in Fig. 47*e*. Sometimes it is placed at the middle of the plate, but in this case the longitudinal shearing surface behind the lug is too small to insure an effective anchorage for the beam. In Fig. 41*a* the beam is held longitudinally by lag screws on the sides, but with this detail the beam is only released by splitting or shearing its end. Instead of a lug to hold the plate in position, the plate is often made trapezoidal in form to fit a corresponding recess in the wall. In both of the preceding figures the wall is arched over the beam in order to secure ventilation around it. Cast-iron plates or flat stones are often substituted for the arch.

The Goetz-Mitchell system of anchoring a beam is illustrated in Figs. 41*c* and *d*. The anchor consists of a cast-iron box of dovetail shape to lock into the wall, and with a lug on the bottom plate over which the joist or beam is notched. For the wider boxes interior side guides are provided which permit a

free circulation of air to prevent dry rot. The end of the beam is cut at an inclination to enable the falling beam to release itself from the anchorage. The box is provided with a cover to support the masonry directly above it. The box anchor prevents fire or smoke from passing through the wall, and also protects the beam from the danger of fire on the opposite side of a party wall or from adjacent flues. The joists upon one side of a party wall can all burn out while those upon the opposite side hold the wall in position. For heavy timbers the bottom of the box is extended to increase its bearing area on the masonry. The projection of the base plate beyond the lug gives additional bearing for the beam and thereby resists the tendency of the lower fibers to split off.



FIGS. 41*c* and *d*. Goetz Box Anchor and Wall Hanger.

The introduction of this system of anchoring beams was recognized as a noteworthy improvement in building construction and an aid in the reduction of fire risk, by the award of a medal and premium by the Franklin Institute in 1891. The value of an effective method of anchorage is also shown practically by insurance companies in allowing reduced rates of premiums.

Wall hangers are also frequently used for the support of beams. As shown in Fig. 41*d* the Goetz wall hanger consists of the beam hanger bearing on anchor plates of cast iron provided with anchoring lugs, made to set level on the wall and to fit the angle of the hanger arms or hooks. The duplex wall hanger differs from the beam hanger in replacing the hollow cylindrical bearing with a projection extending into the wall resembling an I-beam but with a rectangular opening cut out of its web and with its lower flange extended on each side to pro-

vide the necessary bearing on the masonry. The Van Dorn or national wall hanger is made by riveting a beam hanger to a steel plate which is as wide as a brick horizontally, and is then bent up for bonding into the wall. The former also has the plate bent down a short distance on the face of the wall (Fig. 41*e*).

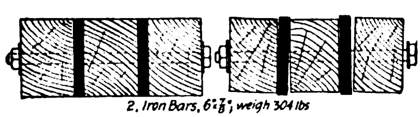
FIG. 41*e*. Van Dorn Wall Hanger.

The use of wall hangers makes the load on the wall more eccentric and the anchorage is less effective than with box anchors. If the beams are fastened too securely to the hangers, the wall will be pulled over by the falling beams in case of fire.

Prob. 41. Find the compression per square inch on the bearing plates at both ends of the beam in Fig. 41*a*. To find the uniformly distributed load, take the allowable stresses in the outer fiber as 1950 pounds per square inch and the unit horizontal shear as 180 pounds per square inch. The distance from the inside of the wall to the center of the column is 18 feet.

ART. 42. COMBINATION BEAMS.

A combination beam is one in which two different materials, as, for example, steel and wood, are combined in its construction. The oldest form of strengthening a wooden beam without changing its form and size, or but slightly increasing the latter,



FIGS. 42*a* and *b*. Flitch-plate Bolsters.

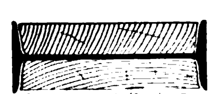


FIG. 42*c*. Bolster.

consisted in inserting an iron plate between two sticks of timber or two plates between three sticks, the timbers and iron plates being thoroughly bolted together so as to act as a single member (Fig. 42*a*). This form is generally known as a flitch beam. A rectangular cross-section is not an advantageous form for metal

to resist flexure, but when the flitch beam was introduced, the modern structural metal shapes were not in existence.

The investigation and design of a flitch beam depends upon the equal deflection of its parts. It is shown in mechanics that for any given span and loading the deflection varies as W/EI in which W is total load, E the modulus of elasticity of the material, and I the moment of inertia of the cross-section of the beam. Since the wooden and steel parts have the same deflection, $W_w/E_wI_w = W_s/E_sI_s$ in which the subscriptions w and s refer to the wood and steel respectively. For rectangular cross-sections of equal depth, the equation becomes $W_w/E_wb_w = W_s/E_sb_s$. Under the same conditions the relation beween the unit-stresses in the outer fiber is

$$\frac{S_w}{S_s} = \frac{W_w}{b_w} \Big/ \frac{W_s}{b_s}.$$

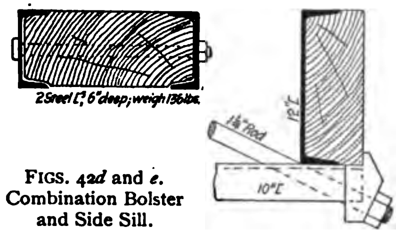
Combining these equations, the following relation is obtained,

$$S_w/S_s = E_w/E_s.$$

For example, if longleaf yellow pine and steel be combined in such a beam, with values of the modulus of elasticity of 1 600 000 and 28 000 000 pounds per square inch, respectively, the timber can be stressed only to 914 pounds per square inch, provided the steel is stressed to its allowable value of 16 000 pounds per square inch. If the steel plate is omitted, the timber can be safely stressed to 1950 pounds per square inch, in the outer fiber, when protected from the weather. This result indicates that under these conditions the efficiency of timber in a flitch beam is only about 47 percent; and since a steel plate has only about 70 percent of the strength of an I-beam having the same depth and sectional area, it is not economical to use a flitched beam when I-beams are available.

Figs. 42*a* and *b* show the sections of a flitch-plate bolster; Figs. 42*c* and *d* of combination bolsters; and Fig. 42*e* of a side

sill in car construction. They represent results of the earlier attempts to strengthen members of the underframe, in order to increase the carrying capacity of modern freight cars. Figs. 42*b*, *c*, and *d* illustrate the effect of shrinkage of the timber. The longitudinal expansion and contraction of the metal, due to changes of temperature, causes additional stresses in their connections as well as in the component parts of the beam. When flitch beams are used in a building, in case of fire the metal plate will twist, and practically destroy the beam.



FIGS. 42*d* and *e*. Combination Bolster and Side Sill.

Prob. 42. Consult MERRIMAN'S Mechanics of Materials, Art. 112, and study the order of design for a flitch beam.

ART. 43. DEEPENED BEAMS.

When the load and span become too large for a single stick of an available commercial size, two general methods may be employed to build up beams of the requisite strength and stiffness. First, by placing side by side two or more sticks of the same depth, but relatively narrow, and bolting them together either with or without intervening spaces; and, second, by placing two or more sticks of the same width on top of each other and bolting them together with keys or brace blocks, as indicated in Fig. 43*b*, to prevent sliding of the contiguous surfaces.

The first type is described in Arts. 38 and 39. The second type is known by various names, including 'built-up beams,' 'compound beams,' or 'keyed beams,' as well as that adopted for the title of this article. Only two sticks are generally employed in deepened beams; although three sticks have been used occasionally, it is more economical in that case to substitute trusses.

If two timbers of the same width b, and the same depth d, are placed on top of each other, as in Fig. 43a, and the friction neglected in the surface of contact, the resisting moment of the timbers is $Sbd^2/3$, in which S is the unit-stress in the outer fiber. If the timbers are so connected by keys or brace blocks, as in Fig. 43b, that they act like a single stick of depth $2d$, the resisting moment becomes $2Sbd^2/3$ thus doubling the combined strength of the separate timbers. The keys must effectively resist any tendency to sliding between the adjacent surfaces, so as to subject all the fibers of the upper timber to compression and those of the lower one to tension. Fig. 43c shows to a large scale the end of a deepened beam, in which the brace blocks are narrower than the beam, the plan and sections of a block being given in Fig. 43d. The position of the end brace block with respect to the center of the support should be noted.

FIG. 43a.
Superimposed Beams.

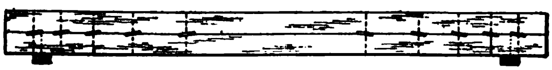


FIG. 43b. Deepened Beam.

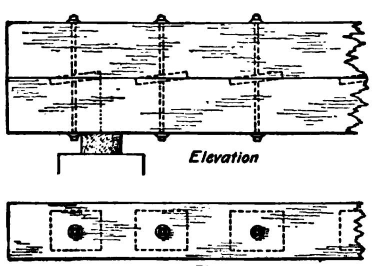


FIG. 43c. Part of Deepened Beam.

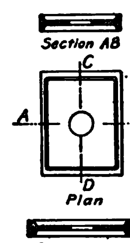


FIG. 43d. Cast Brace Block.

Deepened beams are less frequently employed in this country than abroad, but they may be found in temporary and other timber structures for mining operations remote from large cities,

as well as in railroad beam bridges in localities where timber is cheaper and more accessible than steel.

The following statement regarding deepened beams is made by J. P. SNOW in an article on Modern Bridge Construction on the Boston and Maine Railroad, in Journal of the Association of Engineering Societies, vol. 15, page 31, July, 1895: "These stringers require considerably more material than trusses of equal strength, but the labor on them is small and they can be put in place and prepared for the passage of trains in much less time than trusses. This latter quality is of great importance, and should be given more attention by bridge designers than it generally receives. This style of bridge works in with the ordinary trestle spans very conveniently when it is desired to make a wide opening for a runway for ice or logs, or for a highway underpass. The lower stick of the compound stringer is extended beyond the upper one to furnish a seat for the regular stringers of adjacent bents. In cases, too, where the trestle bents are high and expensive, it will lessen the cost of the structure to make alternate spans compound or keyed stringers. This style of bridge is available up to clear spans of 30 feet."

Prob. 43. Refer to WARREN'S Engineering Construction in Iron, Steel, and Timber, page 92, and copy the illustration of a deepened beam of ironbark wood in which the keys vary in size and are spaced uniformly instead of being uniform in size and spaced unequally.

ART. 44. PRINCIPLES GOVERNING DESIGN.

In the design of a deepened beam the following principles and details should be considered, in order to secure the same degree of safety in all its parts as well as economic construction.

RELATION OF WIDTH TO DEPTH OF BEAM. — At or near the section where the bending moment is a maximum the strength depends upon the stress in the outer fibers, and is therefore

proportional to the width and to the square of the depth. At the ends of the beam the strength depends on the unit stress in horizontal shear at the neutral surface, and hence the strength varies directly as the width and depth. Accordingly, the most economical width is that which gives only sufficient shearing area between the two keys having the least spacing. As, however, the end key may be moved beyond the support without increasing the shear to be resisted by it, the spacing between the second and third keys really determines the relation of width to depth.

Since the length of shearing surface required can only be found after a preliminary determination of the cross-section, the width may be assumed for this purpose as equal to, or a little greater than, the half depth. Instead of assuming the width b, and finding the depth d after the value of bd^2 is known, it is better to assume d first and then find b, revising the result until the dimensions have the relation indicated above.

NUMBER OF KEYS. — The number is to be assumed so as to give a reasonable depth to provide the necessary bearing area on the ends of the keys. It is to be remembered that the cost of workmanship increases with the number of keys, while the difference between the cost of a small and of a large key or brace block is relatively slight. As the number of keys is increased it also requires better workmanship to have them all act together to take their proper share of the horizontal shear. On the other hand, as the number of keys is reduced, the distances are increased through which the stresses must be transmitted from one timber to the other.

It may be added that deepened beams with only two brace blocks in the half span have been used in railroad bridges with a clear span of 20 feet, but this arrangement makes the spacing between the inner blocks about 11 feet, which seems to be too large a part of the span.

DESIGN OF THE BRACE BLOCKS. — The web transmits the resultant compression in its direction from one end flange to the other. The flanges at the sides serve mainly to stiffen the web. The flanges at the ends act as double cantilevers supported by the web, and receive as a uniformly distributed load the compression or bearing of the fibers of the beam. Sometimes the thickness of the web is governed by the minimum thickness allowed by practical considerations in founding rather than by the stress transmitted. The flanges must have a slight bevel on the inside to allow the pattern to be readily withdrawn from the mold. Since the resultants of the horizontal pressure on the ends of the block are eccentric, the block tends to rotate, and this must be resisted by the vertical pressure of the sides of the fibers on the end flanges. In order to keep the length of the brace block within reasonable limits, it may be necessary to thicken the flanges to increase this bearing area. To keep the timbers from separating they are bolted together through the block, the tension in the bolt being equal to the resultant of the vertical pressure on each end of the brace block. If the beam is exposed to the weather, the blocks may be made narrower than the timber, thus preventing water from collecting in the notches, or, it may be protected by a covering of tin properly arranged. In general the length of the key should be nearly equal to its width.

POSITION OF THE BRACE BLOCKS. — Each block can resist only horizontal shear which is developed on its side toward the section where the maximum bending moment is located. The blocks are therefore to be placed with their inner edges at the points which divide the total horizontal shear into equal parts. An error in design which is often noticed consists in placing the end block inside of the center of the support. As the distance between the inner block of each half span is relatively large, it is usually necessary to keep the timbers in close contact by additional bolts, taking care not to locate any bolt so close to the

section of maximum bending moment as to increase the unit-stress beyond its safe value.

Length of the Shearing Surface. — When the keys or blocks are horizontal, as in Fig. 16*a*, there is no doubt as to the length of available shearing surface between the blocks; but when they are inclined, it depends upon the direction of the grain in the timber. It may be close enough for practical purposes to assume that the shearing surface for any block extends from its outer end to the middle of the next block. Since no vertical shear exists beyond the support of a simple beam, no horizontal shear can develop there, and hence the shearing surface between the first and second blocks may be increased if desired by moving out the end block. If any other block be moved, it changes the pressure on it and on the next outer block.

On account of the shrinkage of the timbers as seasoning progresses, and the tendency of the brace blocks to separate them, no reliance can be placed upon the friction between the two timbers to resist any part of the horizontal shear.

Art. 45. Design of a Deepened Beam.

The following order of design is suggested for the guidance of the student: 1. Draw the symbolic loading in position. 2. Construct the bending moment diagram for the external loading on the beam. 3. Construct the vertical shear diagram for the external loads. 4. Find the dimensions of a solid timber to resist the greatest bending moment, remembering the relation between width and depth indicated in Art. 44. 5. Compute the weight of the timber and estimate the additional weight of details, the opposite sides of the axes of the bending moment and vertical shear diagrams; construct the corresponding diagrams due to the weight of the beam. 6. Revise the section of the beam, if its resisting moment is not great enough to include the bending moment due to its own weight. 7. Divide the ver-

tical shear diagram into a series of narrow strips and find the area of each strip. 8. Draw the key diagrams. Assume the number of keys or brace blocks in the half span, and compute the horizontal shear to be resisted by each block. Find the positions of the inner sides of the blocks. 9. Compute the length of shearing surface required for each block. 10. Design a brace block to resist this pressure: (*a*) width of end flange; (*b*) thickness of web; (*c*) thickness of end flanges next to web; (*d*) their thickness at the edges; (*e*) length of the brace block; (*f*) thickness of side flanges. 11. Compute the length of shearing surface required between the blocks. 12. Design the bolts and washers. 13. Determine location of bolt between inner block and position of maximum bending moment. 14. Compute the bearing areas and widths required for the supports.

For example, let it be required to design a deepened beam with a span of 20 feet, to support two concentrated loads, one of 6000 pounds at 7 feet, and the other of 15 000 pounds at 15 feet from the left support, and also a uniform load of 5000 pounds per linear foot extending between two points respectively distant 3 and 13 feet from the left support.

The following working unit-stresses are specified:

	Pounds per square inch.
Shortleaf Yellow Pine	
Extreme fiber stress in bending	1 650
Longitudinal shear	200
Compression on the side of the fibers	250
Compression under washers	275
Compression on the ends of the fibers	1 650
Cast-iron Brace Blocks	
Extreme fiber stress in bending	5 000
Compression	15 000
Steel Bolts	
Tension	15 000

The loading is indicated in Fig. 45*a* in magnitude and position, a scale of 2 feet to an inch being used for the original drawing. The bending moment and vertical shear diagrams are constructed in accordance with the principles of graphic statics. See MERRIMAN and JACOBY'S Roofs and Bridges, Part II, Chap. I and Art. 62. A convenient scale for the load line (not reproduced in Fig. 45*a*) and vertical shears is 10 000 pounds to the inch. By computation the left reaction for the external loads is 37 650 pounds. The pole is taken directly opposite the initial point of the reaction on the load line so as to make the closing line of the equilibrium polygon horizontal. In drawing this polygon, the uniform load on each side of the 6000-pound concentrated load is concentrated at the two respective centers of gravity, and then the two parabolas are drawn tangent to the sides of the polygon at the sections where the corresponding portions of the uniform load begin and end. The pole distance H is laid off by scale to equal 20 000 pounds. The ordinates y are measured by the linear scale and their values multiplied by H, since by graphic statics the bending moment $M = Hy$. If preferred, the moments may be read off directly by using a scale of 40 000 pound-feet to the inch, this moment scale being H times the linear scale.

The greatest bending moment is checked by computation and found to be 237 100 pound-feet, or 2 845 200 pound-inches. For a unit-stress of 1650 pounds per square inch the resisting moment of a beam of breadth b and depth d is $1656\ bd^2/6$ pound-inches. Equating the values of the two moments, $bd^2 = 10\ 346$ inches^3. Assuming $d = 28$ inches, b is found to be 13.2 or 14 inches. At 40 pounds per cubic foot the weight of the beam is 2180 pounds, and in order to make ample allowance for bolts and brace blocks the total weight is assumed to be 2600 pounds. The corresponding bending moment at the middle of the span is 6500 pound-feet. After the moment and shear diagrams are laid off on the opposite sides of the respective axes, so that

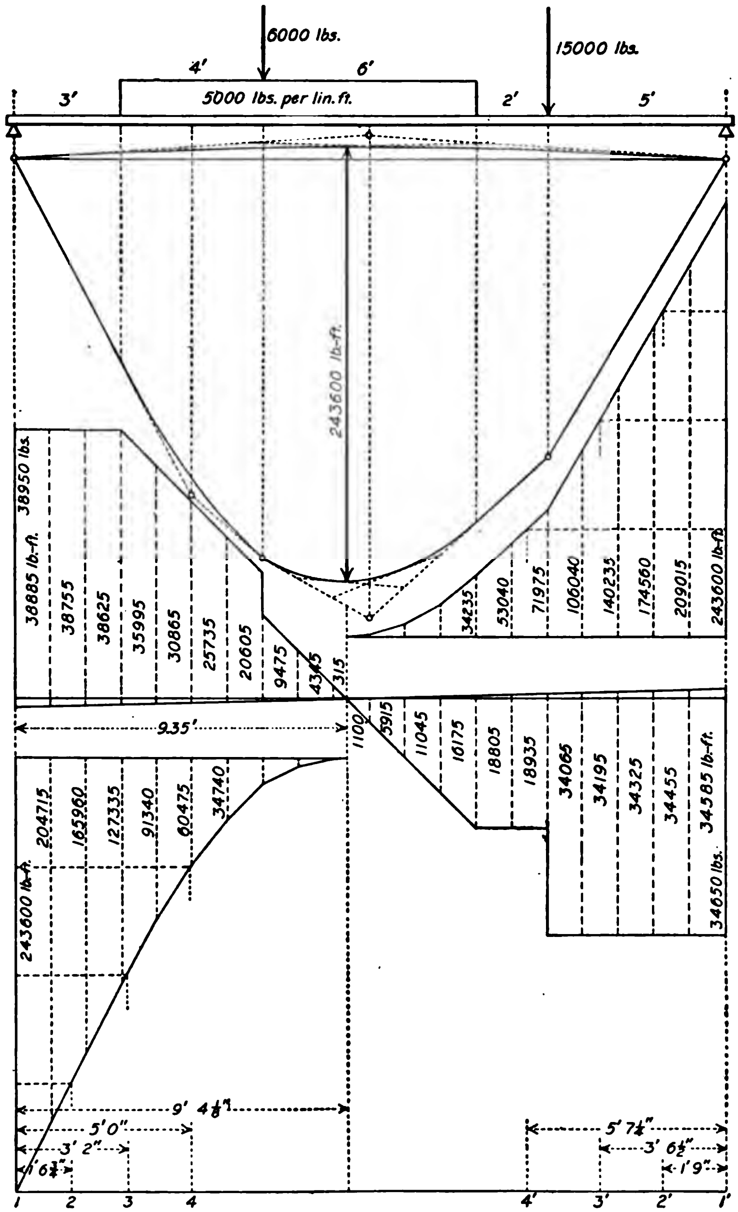


FIG. 45a. The Graphical Analysis of a Deepened Beam.

their ordinates will be added to those previously drawn, it is found that the shear is zero at 9.35 feet from the left support, and the maximum moment is 243 600 pound-feet. The total reaction at the left support is 38 950 pounds. Since $bd^2 = 14 \times 28^2 = 10\,976$ inches^3, the resisting moment is slightly larger than the maximum bending moment and hence no revision of the section is necessary.

The vertical shear diagram is next divided into a series of strips one foot wide by scale, and the values of the shears are marked on the ordinates. The areas of the strips expressed in pound-feet are marked on them. The values of the areas only are given in Fig. 45*a*. A thorough check on the work is based on the fact that the area of either the positive or negative portions of the shear diagram is equal to the maximum bending moment, since according to mechanics $\int Vdx = \int dM$, in which V is the vertical shear at any section of the beam and dM is the moment increment for the differential length dx.

As each brace block is to resist its share of the horizontal shear, it is necessary to construct a diagram which shows the distribution of the horizontal shear at the neutral surface of the beam; but since the horizontal shear is proportional to the vertical shear, the problem of locating the brace blocks may be solved by a proper subdivision of the area of the vertical shear diagram on each side of the zero shear. This is most conveniently done by constructing a key diagram, in which each ordinate represents the area of the vertical shear diagram from the corresponding section to the one where the shear is zero. The key diagram is shown below and above the shear diagram for the two parts respectively. It may be well to call attention to the fact that where the shear diagram is inclined under uniform load that the key diagram is curved.

Let s_h denote the unit horizontal shear, V the vertical shear in any section of the beam, b and d the breadth and depth of its

rectangular cross-section; then, according to mechanics, $s_h = 3V/2bd$. To obtain the horizontal shear for any distance dx along the beam, s_h must be multiplied by the area bdx, giving $s_h bdx = (3/2d)Vdx$. The total horizontal shear between the left support and the section of zero shear is $\int s_h bdx = (3/2a)\int Vdx$. The total horizontal shear in this example is $(3 \times 43\,600 \times 12)/(2 \times 28) = 156\,600$ pounds. Assuming 4 brace blocks for this portion of the beam, each one resists a compression due to the horizontal shear of $156\,600/4 = 39\,150$ pounds.

To find the positions of the blocks, the end ordinate of the key diagram is divided into four parts, and lines drawn parallel to the axis to intersect the curve. It is found that these sections are located respectively 1′ 6¾″, 3′ 2″, and 5′ 0″ from the center of the left support. The corresponding sections in the right portion of the beam are located 1′ 9″, 3′ 6½″, and 5′ 7¼″ from the right support. Since the brace block at the left support may be moved farther to the left if necessary, without changing the pressure which it must resist, the critical spacing is that between blocks 2 and 3. The distance between the sections is $38 - 18.75 = 19.25$ inches. For an allowable unit-stress of 200 pounds per square inch, the net length of shearing surface required is $39\,150/(200 \times 14) = 14$ inches. The difference between these distances is probably sufficient to allow for the notch cut for the block and some lateral deviation of the grain of the timber.

The height of the brace block or width of its end flanges must provide sufficient bearing area for compression on the ends of the wood fibers. For a unit-stress of 1650 pounds per square inch, the width of flanges is $39\,150/(1650 \times 14) = 1.695$, or $1\frac{3}{4}$ inches. Allowing for a 2-inch hole in the web, its thickness must be at least $39\,150/(15\,000 \times 12) = 0.22$ inch; but to secure a good casting of the size of such a block, the web should not be less than $\frac{1}{2}$ inch. Each cantilever of the flange projects $\frac{1}{2}(1.75 - 0.50) = 0.625$ inch beyond the web, and its load is

39 150 × 0.625 / 1.75 = 13 980 pounds. The maximum bending moment is in the section at the web, and equals 4369 pound-inches. Equating this with the resisting moment of the section of the cantilever, the working unit-stress in the cast iron being 5000 pounds per square inch, the thickness of the flange at the web is found to be 0.612 or $\frac{5}{8}$ inch.

Let the length of the block (along the beam) be assumed as 12 inches, and let a width of 2 inches be assumed for the bearing at the flanges to resist rotation, then a sketch of the side elevation of the block indicates a horizontal lever arm for the bearing couple of 9.625 inches (see Fig. 45*b*). The moment of rotation for the block is 39 150 × 1.75 = 68 510 pound-inches,

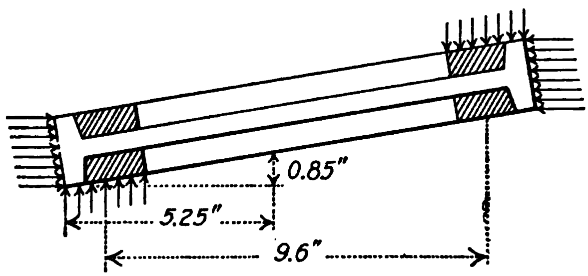


FIG. 45*b*. Pressures on Brace Block.

and the pressure on each side bearing is 68 510/9.625 = 7118 pounds. For a unit-stress of 250 pounds, for compression on the sides of the fibers, the width of bearing required is 7118/(250 × 14) = 2.04, or $2\frac{1}{8}$ inches. It is impracticable to make the flanges $2\frac{1}{8}$ inches thick on the edges, for the metal in the flanges and web would differ so much in thickness as to cause large internal stresses while the casting cools. A better plan is to make the edges of the flanges $\frac{1}{2}$ inch thick at the edges, or $\frac{1}{8}$ inch thinner than at the web, thus allowing the pattern to be easily drawn from the mold, and to insert a wooden strip on each side to supply the balance of the bearing area. The strips can be screwed together in place, if holes for the purpose be left in

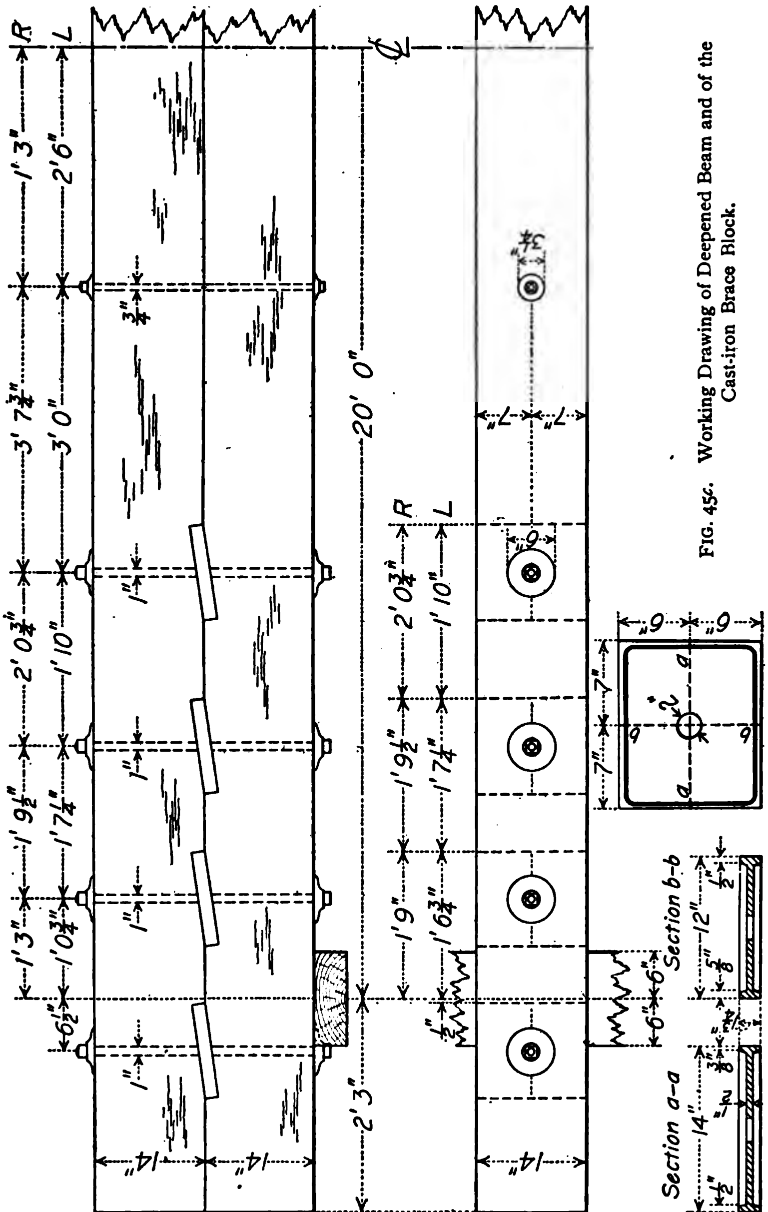


FIG. 45c. Working Drawing of Deepened Beam and of the Cast-iron Brace Block.

the web. The side flanges may be taken $\frac{1}{8}$ inch less in thickness than the end flanges.

Let Fig. 45*b* represent brace block No. 2. The horizontal shearing surface required for No. 3 extends to a line $5\frac{1}{4}$ inches from the lower left-hand edge of No. 1. If a triangular notch be cut for the full depth of the block, the end of the horizontal shearing surface is 0.85 inch below the surface of the notch. If the timber is fairly straight-grained, as it ought to be for deepened beams, this distance will probably be sufficient, but if not, the notch may be cut only to the surface of the web, a deeper notch being then made for the flanges.

The bolt requires an area at the root of the thread of $7118/15\,000 = 0.475$ square inch, and hence its diameter is 1 inch. The washer requires a diameter of 6 inches to provide the needed bearing area of $7118/275 = 25.89$ square inches. The bearing area required for the left support of the beam is $38\,950/250 = 155.8$ square inches, and hence its length along the beam is $155.8/14 = 11.13$ or 12 inches. The washer cuts out some of the bearing area, but the required area still remains. The other bearing is practically the same. Let brace block No. 1 be moved $\frac{1}{2}$ inch to the left, in order to provide the same spacing between Nos. 1 and 2 as between 2 and 3.

If $\frac{3}{4}$-inch bolts be used to connect the two sticks between the inner brace blocks, and the holes are bored $\frac{1}{8}$ inch larger, the resisting moment of the beam is 235 800 pound-feet. This value of the bending moment occurs in sections $7'\ 7''$ and $10'\ 10''$ from the center of the left support, and hence the bolts may be placed anywhere between each of these sections and the nearest support.

The detail drawing of the left half of the beam, with all the required dimensions for the entire beam, as well as that of the cast brace block, are given in Fig. 45*c*. The bill of material is as follows:

2 timbers, 14″ × 14″ × 24′ 6″ [26′ 0″]		849.3 ft. B.M.
8 cast-iron brace blocks, 12″ × 14″		237.0 lb.
8 1″ bolts, 31½″ long		63.4 lb.
2 ¾″ bolts, 31″ long		8.3 lb.
16 O.G. washers, 6″ diam.	48.3	
2 O.G. washers, 3½″ diam.	2.8	51.1 lb.
32 wood screws, No. 24, 1½″ long		

The estimate of cost is:

850 ft. B.M. lumber @ 3¢	$25.50
237 lb. cast-iron @ 2½¢	5.83
72 lb. steel bolts @ 3¢	2.16
51 lb. cast-iron washers @ 2½¢	1.28
32 wood screws	.50
18 hr. labor @ 35¢	6.30
Total cost	$41.57

Attention should be called to the fact that large timbers like those included in the above bill of material are usually sawed to order, in which case they can be cut of the length desired, thus saving about $1.50 in the cost.

The gross weight of the lumber for the length of 24 feet 6 inches is 2670 pounds, and the weight of brace blocks, bolts, and washers is 360 pounds, or the weight of the details is 13.5 percent of the main timbers. For the effective span of 20 feet the weight is 2475 pounds, which is 155 pounds less than the dead load assumed.

Prob. 45. Compute the size of a solid beam of minimum weight, to carry the same loads as the deepened beam used in the previous example, under the assumption that the beam is a single stick, sawed to order, to within a quarter of an inch of each dimension required.

ART. 46. CONSTRUCTION OF DEEPENED BEAMS.

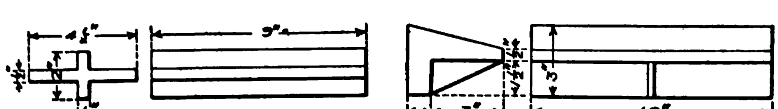
In the construction of deepened beams only timber which is straight-grained should be employed. If in any part of the length the grain is not quite parallel to the horizontal surface, especial care should be taken to cut the notches or indents on

that side where the grain runs away from the surface toward the ends. Unless this precaution is observed, the shearing surface between the keys may be shorter than the length designed.

The seats of the brace blocks in the two timbers are to be so located with respect to each other that when the parts are put together the beam will have a camber. The initial camber must be sufficient to allow for the local adjustment, which brings all the blocks to a solid bearing, and also to prevent the beam from deflecting below the horizontal when under its full load. This is necessary to insure that all the fibers of the lower stick shall be in tension.

Before fastening the timbers together their adjacent surfaces should be painted with hot creosote oil, avenarius carbolineum, or some other preservative in case the beam is to be exposed to the weather (Art. 34). The bolts should be drawn up to their safe stress so that no further elongation occurs when the beam is loaded. If the timber is not seasoned, the bolts will require adjustment afterward.

Fig. 46*a* shows another form of key which has been used in practice. The compression of the horizontal plates on the sides of the fibers varies from zero at the center to the safe unit-stress at the ends. The four arms act as cantilevers, and the thickness of each one near the middle must resist the maximum bending moment. While cast-iron keys of this form have been employed



FIGS. 46*a* and *b*. Special Forms of Keys for Deepened Beams.

in deepened beams used as railroad stringers, it would be safer to use cast-steel keys, since the flexural strength of cast iron is more or less unreliable. The main advantage of this form con-

sists in reducing the loss of horizontal shearing surface to a minimum. Fig. 46*b* gives the form of key used on the Boston & Maine Railroad.

A beam composed of a single timber contains the minimum amount of wood when it is designed so that both the greatest unit-stress in the outer fibers and that in horizontal shear at the neutral surface are respectively equal to the allowable safe values. Any deepened beam of the same strength and kind of wood requires more material, since its width must be greater in proportion to its depth on account of the loss of shearing surface due to the keys.

When metal keys are employed and the beam is constructed with a camber, its stiffness is practically the same as for a single stick having the same outside dimensions. When, however, wooden keys are inserted with the grain across the beam, even when tightened with wedges, it is impracticable to secure the same stiffness as with metal keys, since the wooden keys will compress appreciably when the full load is applied. Any movement of the contiguous surfaces of the timbers increases the deflection. Experiments show that with wooden keys the fibers should be placed parallel to those of the beam. Below the elastic limit there is no danger of the fibers interlacing.

The results of an elaborate series of tests of deepened beams with clear spans of 66 and 90 inches may be found in an article on The Efficiency of Built-Up Wooden Beams, by EDGAR KIDWELL, in Transactions American Institute of Mining Engineers, 1897, vol. 27, pages 732–818, with discussions on pages 970–993. An abstract was published in Engineering News, vol. 37, page 149, March 11, 1897; vol. 39, pages 76, 182, Feb. 3, Mar. 17, 1898.

For an interesting application of deepened beams in the construction of lock gates on the Great Kanawha River see Engineering Record, vol. 40, page 5, June 3, 1899.

ART. 47. TRUSSED BEAMS OR GIRDERS.

When the span is too large for the economic use of a single wooden beam, a trussed beam is sometimes substituted when conditions permit a larger depth of space to be occupied. The simplest form is illustrated in Fig. 47*a*, where the beam is supported by a single strut or king-post at the center. To avoid secondary stresses the center lines of beam, rod, and support must intersect in a point. Two tie-rods are frequently used which support the strut and pass outside of the beam, with nuts bearing on a thick plate of steel at each end of the beam.

FIG. 47*a*. Trussed Beam.

The beam is generally of uniform cross-section, but sometimes it is deeper at the middle as in Fig. 47*b*. In this figure the strut is relatively very short, thus making the beam much

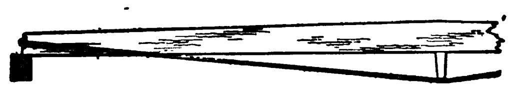


FIG. 47*b*. Trussed Beam.

less effective. Small beams are sometimes composed of two timbers with a single truss rod passing between their ends. In this case the strut is conveniently fastened to the horizontal

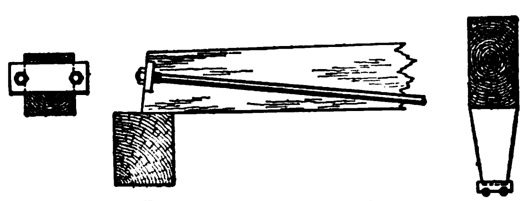


FIG. 47*c*. Enlarged Details.

timbers by a tenon passing between them and connected with bolts or wooden pins. When two rods are used, the beam may

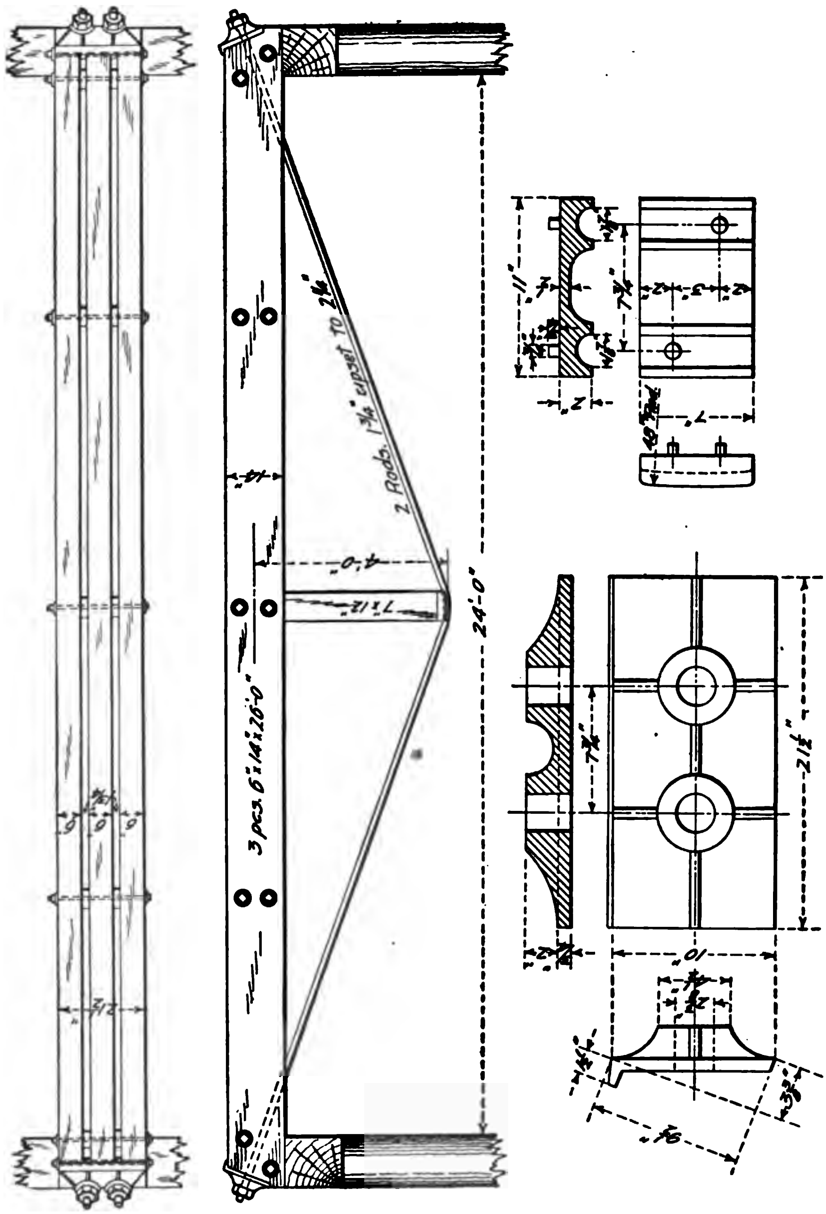


FIG. 47*d*. Trussed Beam on One Side of a Railroad Beam Bridge.

be divided into three sticks, thus allowing the rods to pass between them. This arrangement is shown in Fig. 47*d*. The cast-iron bearing plates and other details indicate that they were carefully designed.

When the trussed beam or girder supports a floor which effectively braces the sticks laterally, the strength is practically the same as if only a single stick is used. When, however, only a concentrated load is supported directly above the vertical strut, the two timbers will act as though they were entirely disconnected between the load and the supports, even though filler blocks may be located at intervals and the timbers bolted together. The unit-stress in compression must then be reduced as in columns for the ratio of the length to the least dimension or least radius of gyration, depending upon the formula adopted.

When two concentrated loads are to be supported, as in the longer spans for which trussed beams are adapted, two vertical struts or queen-posts are used as illustrated in Fig. 47*e*. The

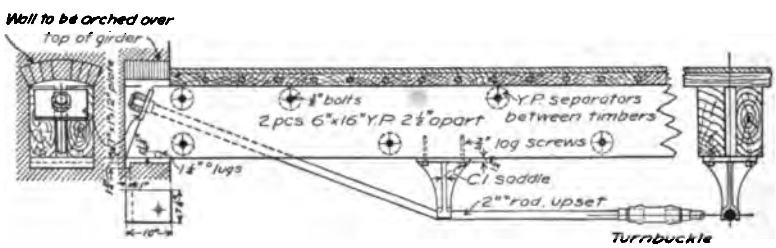


FIG. 47*e*. Details of Trussed Floor Beams for Warehouse.

figure shows a cast-iron strut of star-shaped cross-section. The most common example of the queen-post trussed beam is found in the trussed sills of wooden car frames, illustrations of which may be seen in the Car Builders' Dictionary, or in the volumes of Railroad Age Gazette and other railroad periodicals. In car construction the ends of the rods are flattened and bolted to the sill, and a turnbuckle inserted at the middle of the span to tighten up the rods, or to secure the necessary camber (see Fig. 47*f*).

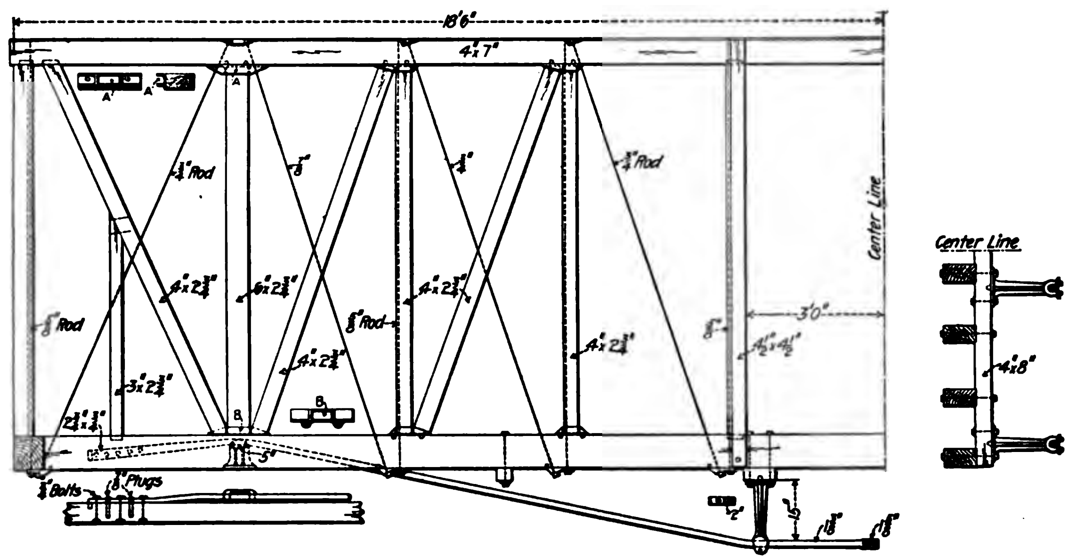


FIG. 47f. Kirby's Framing for a Box Car of 80 000 Pounds Capacity.

This drawing shows the application of truss rods to strengthen and stiffen the sills of the underframe. "The design provides ample supports for the four truss rods 1⅜ inches in diameter over the bolsters and needle beams. The ends are flattened to 2¾ by ¾ inch and secured to the longitudinal sills by two ⅞-inch iron plugs and three ¾-inch bolts, making a strong and simple fastening." — Railroad Gazette, vol. 34, page 961, Dec. 19, 1902.

In the queen-post form the struts are sometimes inclined so as to bisect the angle between the horizontal and inclined portions of the truss rod in order to avoid any tendency to flexure. A beam with this detail used as a floor beam in a conveyor bridge is illustrated in Engineering Record, vol. 34, page 159, Aug. 1, 1896. The span is 18½ feet, the beam is composed of two sticks 4 by 12 inches spaced 2¼ inches apart in the clear, and each strut is 6 by 6 inches. At the upper end a tenon projects between the timbers and is connected by two bolts ⅝ inch in diameter, while at the lower end is a cast bearing block with a projecting dowel. Only one rod is used with a diameter of 2⅛ inches. See also Engineering Record, vol. 23, pages 356 and 376, May 2 and 9, 1891; and Engineering News, vol. 34, page 66 (inset), Aug. 1, 1895.

A novel application of the trussed-beam construction is described in an article on Two 100-foot Spans of Trussed Water Pipe, in Engineering Record, vol. 25, page 111, Jan. 16, 1892. Another one is illustrated in an article on An Ingenious Method of Hauling Heavy Machinery over Country Roads, in Engineering News, vol. 51, page 62, Jan. 21, 1904. A third one has dimensioned detail drawings given in an article on An Adjustable Scaffold for a Paint Shop, in Railroad Gazette, vol. 36, page 348, May 6, 1904.

An interesting comparison between the strength and deflection of trussed beams, flitch beams, and a combination of the two, with a plain wooden beam and a pressed-steel beam, is recorded in Railroad Gazette, vol. 27, page 50, Jan. 25, 1895. The beams tested are in the form of car bolsters, supported near the ends and loaded at the center.

ART. 48. DESIGN OF TRUSSED BEAMS.

On account of the continuity of the beam from one support to the other, the true method of determining the stresses in all

the members of a trussed beam requires the aid of the principle of least work. If a single load be placed at B in Fig. 48*a*, some part of it will be carried by the timber AC acting as a simple beam, while the rest of the load will be carried by the truss of which the beam forms the upper chord. According to the principle of least work, the load is divided so as to make the total

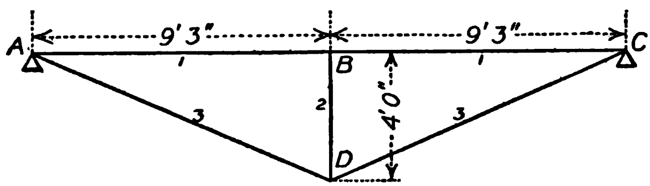


FIG. 48*a*.

internal work of all the stresses in the members a minimum (see Roofs and Bridges, Part I, Chap. VII). It also follows that the division of the load is such as to cause the same deflection of the point B in the simple beam as in the truss.

To find the stresses by means of the method of least work requires the section areas of the members to be known. It is therefore necessary to find these approximately, and then to investigate the structure to learn if any of the stresses exceed the specified limit. For this purpose let it be assumed that the compression in the strut BD is the same as if the beam AC were discontinuous at B.

For example, let it be required to design a trussed beam for a building, of the span and depth indicated in Fig. 48*a*, to support a uniformly distributed load, including its own weight, of 1800 pounds per linear foot. The beam is held in position laterally by the flooring. With the aid of a table of squares (Art. 84), the length of the diagonal AD or DC is found to be 10.078, or 10 feet $0\frac{15}{16}$ inch. If θ is the angle which the diagonal makes with the vertical, $\tan\theta = 9.25/4 = 2.3125$ and $\sec\theta = 2.5197$. The load at B is $1800 \times 9.25 = 16\,650$ pounds, and the reaction of

the truss due to this load only is 8325 pounds. The tension in the diagonal is 8325 $\sec\theta = 21\,000$ pounds. The compression in the vertical is 16 650 pounds, and the compression in the upper chord is 8325 $\tan\theta = 19\,250$ pounds.

The beam and strut are to be longleaf yellow pine, with the following allowable unit-stresses expressed in pounds per square inch:

Extreme fiber stress in flexure, 1950; longitudinal shear, 180; compression on the side of the fibers, 390 pounds; for columns under 15 diameters, 1470 pounds; for columns over 15 diameters, $1950\,(1 - \frac{1}{60}\,l/d)$; and modulus of elasticity, 1 200 000. The diagonal rods are to be of soft steel, the allowable tensile stress being 15 000, and the modulus of elacticity 26 000 000. The modulus of elasticity for the yellow pine is taken less than its average value as derived from full-size tests (Art. 82), since part of its load is fixed or permanent.

The left half of the beam, or AB, is supported at its left and fixed at its right end, the maximum bending moment, which occurs at B, being negative. Its value, according to mechanics, is $-16\,650 \times 111/8 = -23\,100$ pound-inches. Assuming the depth of the beam to be 12 inches, the resisting moment is $1950\,b\,(12)^2/6 = 46\,800\,b$. Equating the two moments, b is found to be 4.936 inches.

In order to allow for the direct compression, let the width be taken as 6 inches. The unit-stress due to flexure is then $S' = 231\,000 \times 6/(6 \times 12^2) = 1604$. The unit-stress due to direct compression at B is $S'' = 19\,250/72 = 267$, making the total unit-stress $S = 1871$ pounds per square inch. Therefore, the beam section may be taken as 6 by 12 inches.

The section area needed by the strut to resist its own stress is $16\,650/1470 = 11.33$ square inches, which is given by a 3 by 4-inch stick. But 3 inches is only 1/16th of the length of the strut, and hence the average unit-stress is only 1430 pounds,

making the revised area 11.65 square inches. But the bearing of the strut on the bottom of the beam really governs its size, unless a bearing plate be provided. The bearing area required is 16 650/390 = 42.72 square inches, and hence a 6 by 8-inch strut will be adopted.

The required section area of the tie-rods is 21 000/15 000 = 1.40 square inches. On account of its length, it is most economical to use a rod with upset ends, hence the diameter required is $1\frac{3}{8}$ inches, providing a section area of 1.485 square inches. The diameter of the upset end is $1\frac{3}{4}$ inches, according to the manufacturer's handbook..

Let w_1 be the portion of the load per linear inch carried by the simple beam AC; its deflection at B being $\Delta = 5\ w_1\ (222^4)$ $/(38 \times 1\ 200\ 000 \times 1008 = 0.02614\ w_1$ inches. Let w_2 be the portion of the load per linear inch carried by the truss, let P be the load carried to the panel point B, and which causes its deflection. According to the method explained in Roofs and Bridges, Part I, Art. 86, the deflection of B is next computed.

MEMBER.	S POUNDS.	l INCHES.	A SQ. INS.	E POUNDS / INCHES².	$\frac{S^2l}{AE}$
Strut BD	$-$ 1.000 P	48	48	1 200 000	0.00000083 P^2
Chord AC	$-$ 1.156 P	222	72	1 200 000	0.00000343 P^2
Tie AD	+ 1.260 P	120.94	1.485	2 600 000	0.00000416 P^2
Tie DC	+ 1.260 P	120.94	1.485	2 600 000	0.00000416 P^2

The sum of the products in the last column is 0.0000126; then the deflection is

$$\Delta_2 = \frac{1}{P}\sum\frac{S^2l}{AE} = 0.0000126\,P,$$

and since $P = 111\ w^2$, the deflection is $\Delta_2 = 0.00140\,w_2$. Equating the two deflections and remembering that $w_1 + w_2 = 150$ pounds, there is found $w_1 = 7.63$ and $w_2 = 142.37$ pounds per

linear inch, making $P = 15\,803$ pounds per square inch. The deflections Δ_1 and Δ_2 are 0.1992 inch.

The tension in each tie-rod is 19 910 pounds, and the unit-stress 13 275 pounds per square inch. The bending moment in the wooden beam at B, under the assumption that the truss does not deflect at all, is $M' = -\frac{1}{8} \times 150 \times 111 \times 111 = -231\,000$ pound-inches. But since it actually deflects the same amount as by carrying a load of 7.63 pounds per linear inch, acting as a simple beam with a span of 222 inches, a positive bending moment is developed at B of $M'' = \frac{1}{8} \times 7.63 \times 222 \times 222 = 47\,000$ pound-inches. Accordingly, the true bending moment at B is $M = -231\,000 + 47\,000 = 184\,000$ pound-inches. The stress per square inch in the outer fiber due to flexure is $S' = 6 \times 184\,000/(6 \times 12^2) = 1278$ pounds, and that due to the direct compression is $S'' = 18\,270/72 = 254$ pounds, making $S = S' + S'' = 1532$ pounds, which is less than the limit specified.

Let an investigation be made next to see what the greatest unit-stress is at the section where the positive bending moment is a maximum. The moment diagram is given in Fig. 48*b*. The curve for the beam AC is laid off below the axis in order to add its ordinates algebraically to the others.

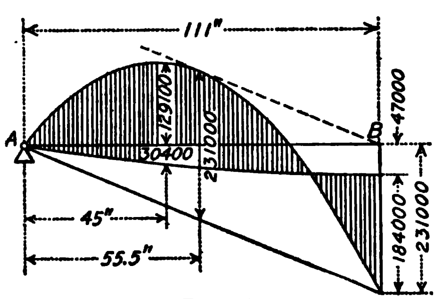


FIG. 48*b*.

The general expression for the moment curve above the axis is $M' = \frac{3}{8} \times 150 \times 111x - \frac{1}{2} \times 150x^2$, and that for the curve below the axis is $M'' = \frac{1}{2} \times 7.63 \times 222x - \frac{1}{2} \times 7.63x^2$, and of their sum is $M = M' + M'' = 7091x - \frac{1}{2} \times 157.63x^2$. Placing the first differential coefficient equal to zero, there is found $x = 45.0$ inches, and $M = +159\,500$ pound-inches. It is next required to find the greatest stress in

the outer fiber due to the bending and the direct compression combined. For this purpose the formula

$$S' = Mc/\left(I - \frac{n}{m} \cdot \frac{Pl^2}{E}\right)$$

is employed, in which c is the distance from the neutral axis to the outer fiber, I the moment of inertia of the section, P the direct compression, l the length, E the modulus elasticity of the material, while m and n are constants relating to the strength and stiffness of beams, the values of which depend upon the loading and the condition of the ends of the beam (see MERRIMAN'S Mechanics of Materials, Art. 102). The beam AB (Fig. 48a) under consideration is uniformly loaded, supported at A and practically fixed at B, and for these conditions $n =$ 0.0054 and $m = 9/128$, or $n/m = 1/13$. Accordingly the maximum stress in the outer fiber, expressed in pounds per square inch, is

$$S' = 159\,500 \times 6/\left(864 - \frac{18\,270 \times 111 \times 111}{13 \times 1\,200\,000}\right) = 1127,$$

and the direct compression is $S'' = 18\,270/72 = 254$, making the total unit-stress $S = 1381$ pounds per square inch, which is less than that at B. To ascertain whether the beam may be reduced to 5 by 12 inches will be left as an exercise for the student.

In some cases it may be necessary to build the trussed beam with an initial camber. For example, let a camber of $\frac{3}{16}$ inch be put into the beam under consideration in this article. Let w_3 be the load per linear inch required to deflect the truss a distance equal the camber; then $0.0014\,w_3 = \frac{3}{16}$, whence $w_3 =$ 133.93 pounds. The remainder of the uniform load of 150 pounds, or 16.07 pounds, is divided between the beam and the truss in the same manner as before, giving $w_1 = 0.82$, $w_2 =$ 15.25, and $w_2 + w_3 = 149.18$ pounds, making $P = 16\,559$ pounds. The direct compression in AB is 19 140 pounds, and the tension in the rod is 20 860 pounds. $M'' = 0.82 \times 222 \times 222/8 =$

5100 pound-inches, and since $M' = -231\,000$ as before, $M = -225\,900$ pound-inches. Finally, $S = S' + S'' = 1569 + 266 = 1835$ pounds per square inch. The unit-stress in the rod is $20\,860/1.485 = 14\,050$ pounds per square inch.

When the members AB and BC of a trussed beam are not stayed laterally, the column formula must be applied to obtain the greatest unit-stress due to their compression as members of the truss. The solution of the problem is much simpler when only a concentrated load is to be supported at B, since the bending moment in that case is always a maximum at B.

Those having no experience in using the method of least work should be very careful in its application before making a thorough study of the principles involved by means of standard works on mechanics and bridges. In Statically Indeterminate Stresses in Frames commonly used for Bridges by ISAMI HIROI, may be found a treatment of the king- and queen-post trussed beams as well as of beams reënforced by angle struts and straining beams, supported either by masonry or by timber columns, thus forming a braced bent. A numerical example is solved for each case.

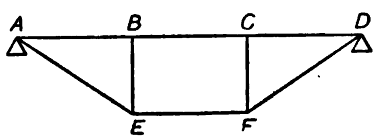


FIG. 48c.

Prob. 48*a*. Design a trussed beam like Fig. 48*a* with the same span and depth, to support a load of 180 000 pounds applied at B, in addition to its own weight.

Prob. 48*b*. The span of the queen-post trussed beam in Fig. 48*c* is 24 feet and its depth is 4 feet. Design the trussed beam to carry a load of 1800 pounds per linear foot, which includes its own weight.

ART. 49. DESIGN OF WOODEN COLUMNS.

According to full-size tests wooden columns generally fail by lateral deflection or buckling, when the length exceeds about twenty times the least dimension of the cross-section. In prac-

tice, however, the unit-stress for all columns over fifteen diameters in length should be reduced in accordance with the various rules or formulas established for long columns. The same unit-stress is employed for all shorter columns as for those having the ratio of slenderness of $l/d = 15$, in which l is the length, and d the least dimension or diameter, both being expressed in inches.

Since most wooden columns are rectangular or square in section, it is customary to employ column formulas containing the ratio l/d rather than the more general form containing the ratio l/r, the term r designating the least radius of gyration of the cross-section. The data which is given when a column is to be designed consists of the axial load, its length, and the condition of its ends. The column formula to be used is either given in the specifications or must be selected by the designer. No column should have a ratio of l/d greater than 60.

As the section area depends upon the average unit compressive stress, and this in turn depends upon the ratio of the length to the least dimension of the cross-section, an approximate section area must first be found in order to estimate closely the least dimension of the column. For example, let it be required to design a rectangular column, 11 feet long, to support an axial load of 28 450 pounds, the column formula specified being $P/A = 1800 - 30\ l/d$, in which P is the axial load and A the section area. For $l/d = 15$ or less, the average compressive stress or unit load is 1350 pounds per square inch, and the section area is $28\,450/1350 = 21.1$ square inches. The least side may accordingly be assumed as 5 inches. For the length of 11 feet, $l/d = 132/5 = 26.4$; $P/A = 1800 - 30 \times 26.4 = 1008$ pounds per square inch, and $A = 28\,450/1008 = 28.2$ square inches. The section required is therefore 5 by 6 inches, no revision being necessary since the least dimension agrees with that assumed.

The theory of columns is not upon such a satisfactory rational basis as that of beams, and in consequence a large number of column formulas of different types have been deduced and are in practical use. The discussion of theoretic column formulas belongs to works on mechanics, and the student is referred to MERRIMAN'S Mechanics of Materials, Chap. IX; CHURCH'S Mechanics of Engineering, Chap. VI; and to other standard works.

Uneven end bearings and the eccentric loading of columns produce more serious effects than are ordinarily assumed in practice, and careful attention should be given by the designer to the actual conditions under which any given column is to be constructed and used. When a column is composed of separate sticks and not connected in a manner to prevent effectively any longitudinal shear between them, it is to be designed as if the sticks were acting independently of each other. This subject is discussed further in the next article.

The subject of columns has been extensively discussed, both theoretically and practically, by engineers and investigators, and the study of the most important articles will serve an important purpose in aiding the designer to judge of the relative importance of various elements in the practical conditions to which columns are subjected, and to secure improved conditions to promote the economic and effective use of the material employed in their construction. For this purpose the following references are given:

The Theory of the Ideal Column, by HENRY S. PRICHARD. Engineering News, vol. 37, page 277, May 6, 1897. See also the editorial on page 280, and discussions on pages 308–311, 329.

An Investigation of the Strength of Columns Leading to Some New Formulas, by CARL G. BARTH. Journal Association of Engineering Societies, vol. 20, page 239, April, 1898.

Theory of the Ideal Column, by WILLIAM CAIN. Transactions American Society of Civil Engineers, vol. 39, page 96,

June, 1898. The theory of the ideal column eccentrically loaded is given in the discussion by A. MARSTON.

The Practical Column under Central or Eccentric Loads, by J. M. MONCRIEFF. Transactions American Society of Civil Engineers, vol. 46, page 334, June, 1901.

Column Formulas, by B. R. LEFFLER. Engineering Record, vol. 50, page 120, July 30, 1904.

Column Formulas in Relation to the Practical Column. An editorial in Engineering News, vol. 57, page 15, Jan. 3, 1907.

Stresses in Simple Columns under Eccentric Load, by O. H. BASQUIN. Engineering News, vol. 57, page 420, April 18, 1907.

Safe Stresses in Steel Columns, by J. R. WORCESTER. Transactions American Society of Civil Engineers, vol. 61, page 156, December, 1908.

The Strength of Compression Members, by EDWARD GODFREY. Railroad Age Gazette, vol. 47, page 18, July 2, 1909.

Prob. 49. Refer to the article in Engineering News, vol. 57, page 15, and compute the equivalent eccentricity for the column in the example given in that article.

ART. 50. CONSTRUCTION OF POSTS.

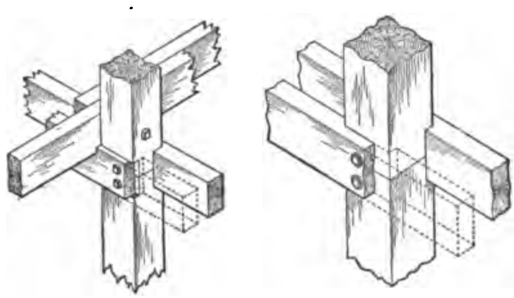
To reduce the tendency of large timbers to check when used as posts in buildings, it is customary in many cases, especially in slow-burning or mill construction (Art. 72) to bore a $1\frac{1}{2}$-inch hole longitudinally through its center, and to provide for the circulation of air through it by means of $\frac{1}{2}$-inch vent holes at the top and bottom. Especial care should be exercised to secure perfectly square ends for bearings in order to avoid secondary bending moments due to the eccentric application of loads.

In practice, it is sometimes considered that knots are not as objectionable in columns as in beams, but this opinion is not confirmed by full-size tests. The presence of a knot involves curved grain around it, and to some extent the combined effect

of compression on the side of the fibers and tension across the fibers. The strength of wood in both of these respects is relatively very low, and hence large knots or smaller knots in groups should be regarded as a sufficient cause for the rejection of timber for columns. Obliqueness of grain also materially reduces the strength of columns and should be avoided. The inspection of timber is discussed further in Art. 78.

The largest single sticks which have been used as columns in this country were 96 feet high and 2 feet in diameter from the middle to the bottom. The load to be supported by each stick was over 90 000 pounds. They were of Douglas fir from the state of Washington and were employed to raise the granite column of the Cathedral of St. John the Divine in New York. See Engineering News, vol. 52, page 183, Sept. 1, 1904.

Fig. 50*b* shows an intermediate joint of a trestle post which differs from the ordinary practice in trestle construction, and



FIGS. 50*a* and *b*. Intermediate Joints of Trestle Posts.

secures a direct bearing between the ends of the fibers, instead of interposing an intermediate cap between them, which limits the strength to that of bearing on the side of the fibers. The posts and batter posts are without mortise, tenon, or notch. The intermediate cap is replaced by two lighter timbers known as a double or split cap, which are notched over the post, while the usual sway braces, and the longitudinal struts and girts and

X-braces, hold the joint in position. In Fig. 50*a* the post is notched as well as the transverse timbers, reducing the section of the post about one-third, but still leaving sufficient bearing area to resist the stress. See Engineering News, vol. 20, page 50, July 21, 1888; vol. 26, pages 312 and 363, Oct. 3 and 17, 1891.

In the report of a Committee of American Railway Superintendents of Bridges and Buildings is contained the following statement: One good method of obviating many of the defects in the mortise-and-tenon joints is by the use of double caps, posts, and sills. That is, instead of using 12 by 12-inch timbers, two timbers 6 by 12 inches can be used, and being properly fitted and securely bolted together, not only give good results as to strength and durability, but expose all defects frequently found in the center of large timbers. Also, being of only one-half the size and weight, they can be handled much more rapidly and consequently much more cheaply and render better access for renewals. Especially is this the case where trains are to be carried during renewals. This method of constructing timber trestles affords greater economy in renewals and has all the advantages of the ordinary mortise-and-tenon construction.

This construction may be still further improved by placing the joint above the double cap. By placing plain fish plates on all four sides, a stronger and more rigid joint can be made than that shown in Figs. 50*a* and *b*. Where still greater strength and stiffness is required, as in splicing the boom of a derrick, a very effective joint may be made by chamfering the edges so as to fit accurately the faces of steel channel fish plates, which are then bolted securely in place. Such a joint is economical and quickly framed.

Fig. 50*c* illustrates the general type of laced or trussed column which has been extensively used in exposition buildings, where very high columns are often required. The main section con-

sists of four heavy timbers of square cross-section, say 12 by 12 inches, set in the four corners of a square, with a clearance between them nearly or quite equal to the thickness of each stick. All four sides are then laced with 2 by 12-inch plank

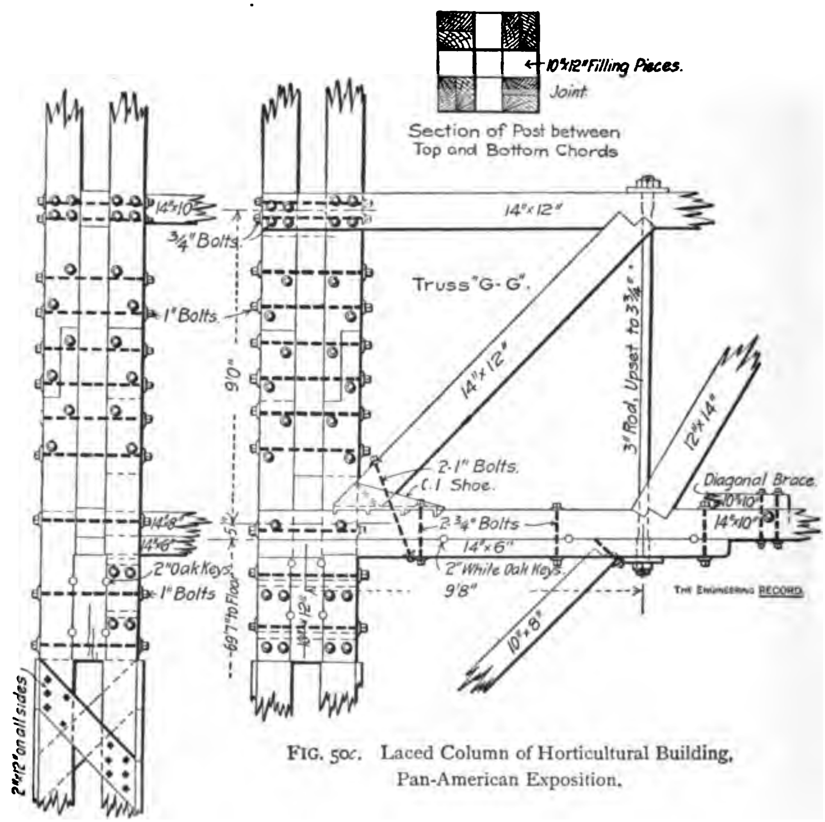
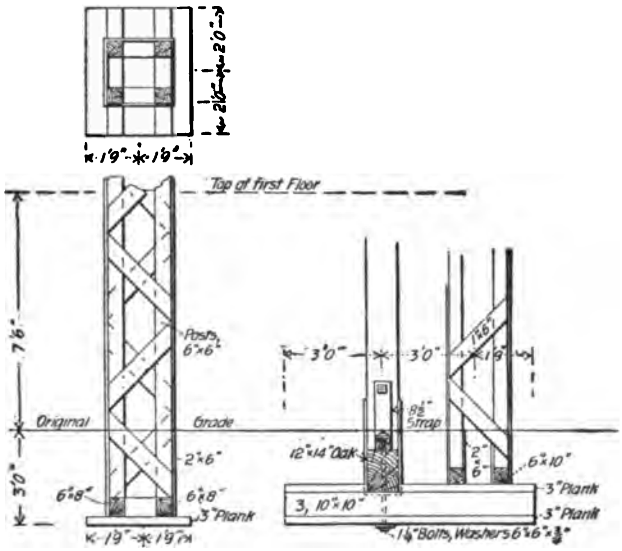


FIG. 50c. Laced Column of Horticultural Building, Pan-American Exposition.

inclined at approximately 45 degrees, except at the connections with stiffening trusses, which are placed at the height of different stories and connect a series of posts.

Where trusses, girders, or struts are supported from the sides of the timbers, seats are provided for them on the upper ends of short vertical planks bolted to the face of the column and keyed

to it by 2-inch round oak pins driven into holes which are bored half in the plank and half in the column. The drawing also



FIGS. 50*d* and *e*. Laced Columns and Column Footings, Machinery Building, Columbian Exposition.

shows the standard construction and splicing of a main column with half-lap joints and with fillers between the timbers, all securely bolted together. It also shows the connection of a stiffening truss which is supported between the heavy timbers composing the column, and indicates the manner in which most of the truss diagonals are stepped into the chord timbers. A sectional elevation showing the general arrangement of columns and stiffening trusses may be seen in Engineering Record, vol. 44, page 182, Aug. 24, 1901.

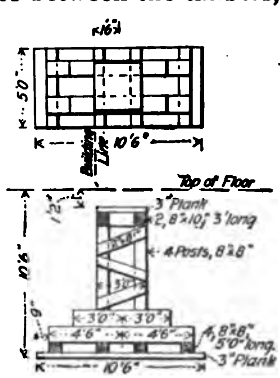
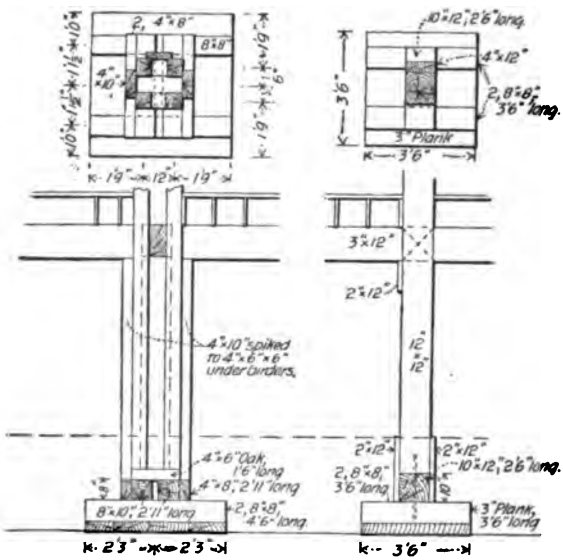


FIG. 50*f*. Column Footing.

Figs. 50*d*, *e*, and *f* represent laced columns used in the Machinery Building of the World's Columbian Exposition in 1893. The lacing plank was spiked to the four sticks composing the column. By placing the timbers in the four corners of a square and effectively lacing them, a column may be economically built with a large radius of gyration. Fig. 50*g* shows a column in



FIGS. 50*g* and *h*. Column Footings, Columbian Exposition.

which the four corner sticks are united by vertical planks spiked to them on three sides. The two 4 by 10-inch planks are also employed to take some beam load directly. Figs. 50*d*–*i* also show various typical forms of footings designed to distribute the column loads on the sandy bottom, with clay substratum, of the exposition site.

When two sticks only are laced together, they are placed with their diagonals in line and spaced far enough apart so that the lacing on both sides can pass between them. In heavy construction the lacing plank are bolted to the sticks.

Composite columns are those in which two or more sticks are bolted or keyed together. In Tests of Metals, 1882, page 239, are recorded the results of tests of such columns composed of two or three sticks of white and yellow pine. In no case did they show materially greater resistance than if each stick were acting freely. The fastenings failed to prevent the slight initial

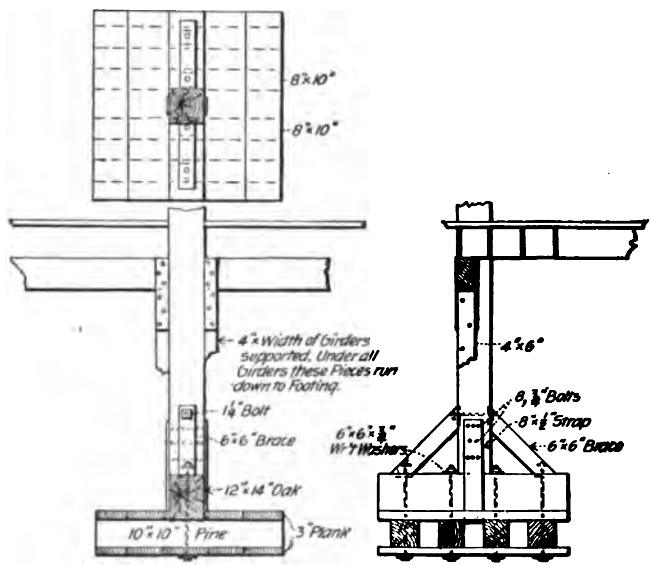


FIG. 50*i*. Braced Column Footings.

lateral deflection. Since the construction employed fails to make the separate members act as a single solid stick, composite columns should not be designed under that assumption. See Art. 6 on the lateral resistance of nails, and an article on Composite Timber Columns by H. P. GILLETTE, in Engineering News, vol. 45, page 439, June 13, 1901.

Prob. 50. Consult the reference to Tests of Metals given in this article and compare the strength of composite and ordinary columns of both white and yellow pine wood by plotting the ultimate loads as ordinates with the ratios of l/d as abscissas.

ART. 51. BOLSTERS.

A bolster is a horizontal cap piece upon the top of a post to lengthen the bearing of a beam supported by it (Fig. 39*d*). The bearing of the beam is on the side of its fibers, while that of the post is on the ends of its fibers. When the beam consists of soft wood, a bolster of hard wood is inserted on top of the post. In Fig. 51*a* large washers or separators are shown between the bolster and beams, apparently to secure ventilation and the drying out of moisture, but this is done at the serious sacrifice of bearing area.

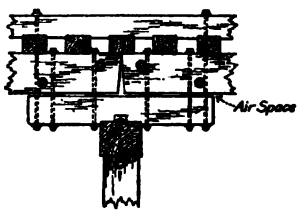


FIG. 51*a*. Bolster.

When bolsters are used, the effective span assumed in computing the strength of stringers in trestles is sometimes taken shorter than the distance center to center of bents, but this is not warranted by actual conditions. Since the stringers generally decay first and most rapidly at their ends, the effective span then becomes less, but the presence of the bolster permits the beam to be kept in service longer than without it. To reduce the strength of the beam, however, at the beginning simply shortens its period of usefulness.

Besides affording better bearing to the stringers at trestle bents, it aids in uniting the ends of the abutting stringers, and materially stiffens the joint, resisting the churning action due to the repeated application and removal of the live load. In case a bent is washed out, the bolster which is bolted to each stringer acts like a fish plate to assist the splice plate in holding the track from collapsing. Experience has shown that accidents have been prevented by this means. (See Engineering News, vol. 40, page 357, Dec. 8, 1898.)

Objections are frequently made to bolsters on account of the increased cost in labor, lumber, and iron, and that additional

places are afforded for decay to begin. It is also claimed that the bearing area is not actually increased, since the deflecting stringer brings the load on the end of the bolster. The last objection may be met by sloping the surface of the bolster slightly toward the ends, if this is deemed necessary.

The importance of providing adequate bearing surface for posts and avoiding the use of soft wood for supports which receive heavy concentrated loads on the side of the fibers, are shown in an instructive account of the Collapse of the Floors of a Warehouse in Cincinnati, in Engineering Record, vol: 49, page 490, April 16, 1904. A number of illustrations of the use of bolsters are given in Art. 53.

The term 'corbel' is frequently used interchangeably with bolster in railroad construction, but the practice is objectionable, since this designation has been applied for centuries to a different detail of construction in architecture and building, namely, "One of a series of brackets, often ornamental, projecting from the face, especially the external face of a wall; used for support, as of a cornice or string course."

When the end of a post is connected to a beam, cap, or sill, by means of short pieces of plank which are spiked and bolted to both members, the joint is known as a plaster joint (Fig. 51*b*). It is used occasionally in trestle construction, but more frequently in temporary framing of various kinds, especially in falseworks. For a typical illustration of its use in falsework for the erection of a bridge, see Engineering Record, vol. 50, page 271, Sept. 3, 1904.

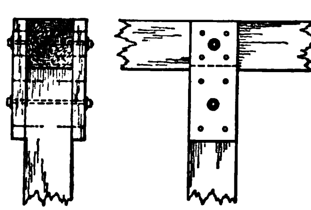


FIG. 51*b*. Plaster Joint.

Eleven tests of columns with bolsters were made in 1894 in the engineering laboratory of the Massachusetts Institute of Technology. There were 8 spruce and 3 oak columns; and 1

yellow pine, 2 maple, and 8 oak bolsters. In nearly every case the column deflected in a diagonal direction and split at the end next to the bolster. The average resistance of the bolster was about 87 percent of the average ultimate strength of the columns. The minimum ultimate resistance was 66 percent of the strength of the column. On the average the spruce columns with bolsters developed 73.4 percent of the strength of the columns without bolsters.

ART. 52. POST CAPS.

In buildings having several stories with plastered partitions and ceilings, the wooden bolster is objectionable on account of the transverse shrinkage of wood due to seasoning, which causes the settlement of floors and cracks the plaster. Uneven resistance in the bolster also reduces the strength of the column by causing eccentricity of loading. The natural remedy was to substitute cast-iron bearing plates or caps. The settlement of floors, however, was chiefly due to the practice of setting the posts of one story on top of the beams which were laid on the post of the next story below. This arrangement was continued even after cast-iron bearing plates were introduced. The next step in advance was to set the post of the next story above directly on the bolster, or the metal plate or cap, and to support the beams on the projecting ends of the latter. This required the caps to resist bending, although the beams had a narrow bearing directly above the lower column, on account of the smaller size of the upper column. The addition of vertical sides afforded the needed flexural strength for the caps.

The introduction of the cast-iron pintle marked the next improvement, and in a modified form is still in extensive use in slow-burning or mill construction. The pintle transmits the load from one column directly to the next lower one, and having a comparatively small horizontal section above its base plate allows the necessary bearing for the adjacent beams. In all the pre-

ceding methods the beams were bolted to bolsters or caps, or fastened together by horizontal straps and other devices, which in case of fire led to serious damage, by pulling down the columns and the floors above. This difficulty was overcome by casting a lug near each end on the bearing plate, which engages a notch in the bottom of the beam, and thus serves to tie the beams together longitudinally, and in case of fire, releasing them without injury to the posts. One form of such a cap, which is still in use, is illustrated in Fig. 41*b*.

To perfect the cast-iron post caps it was necessary to extend the sides both up and down so as to bolt them to the top and bottom columns. This arrangement secures a continuous vertical line of posts with sufficient lateral strength to avoid being readily displaced. The ends of the posts bear directly on the horizontal plate of the cap. These features, together with those

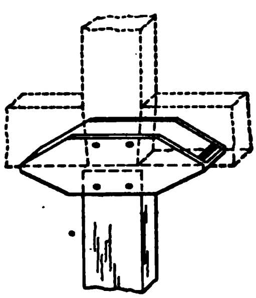


FIG. 52*a*. Goetz Post Cap.

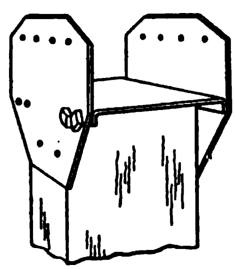


FIG. 52*b*. Duplex Post Cap.

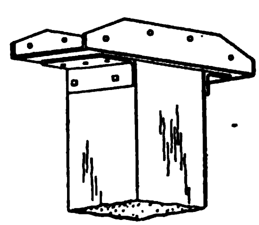


FIG. 52*c*. Goetz Steel Post Cap.

noted in the preceding paragraph, are all embodied in the Goetz post cap (Fig. 52*a*) which was introduced about 1884, and marked a decided advance in building construction which was promptly recognized by Associations of Fire Underwriters.

Other forms of patented post caps in pressed steel followed later, as the demand for better construction increased. The duplex steel post cap (Fig. 52*b*) was placed on the market in 1902, and the malleable-iron cap in 1907. The Goetz steel post

cap is illustrated in Fig 52*c*. Other forms are known as the Van Dorn and national post caps.

Fig. 52*d* shows a steel cap and bracket which was designed in 1897, and since then has been used in some mill building construction. It is intended especially for the connection of beams to wooden columns which are two stories or more in height. It involves very little cutting of the timber and provides for the release of falling beams in case of fire so as not to

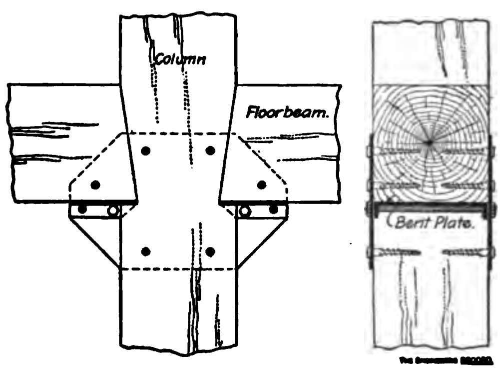


FIG. 52*d*. Beam Seat on Continuous Column.

endanger the stability of the column and other parts of the framework supported by it. Narrow seats, with areas proportioned to the loads on the beams, are cut in the sides of the columns to fit the beveled ends of the wooden floor beams. Horizontal shelf plates are set on them which project far enough to provide sufficient bearing area on the side of the fibers of the beam, and the ends of which are bent down at right angles and bolted to the vertical side or bracket plates, which in turn are fastened to the post by lag screws as shown. The outside bolt on each side of the column passes clear across through both side plates and the vertical ends of the shelf plate, and supports on its side the horizontal plate. The beam is held in place by a single lag screw through each side plate,

which is expected to be torn out without injury to the post in case the beam burns through and falls. This post cap was described in Engineering Record, vol. 43, page 307, March 30, 1901. See also Beam and Column Details in a Factory Building, in Engineering Record, vol. 43, page 526, June 1, 1901.

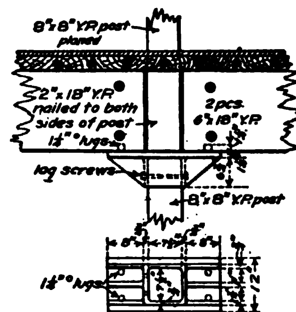
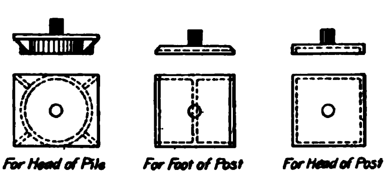


FIG. 52e.

Fig. 52e indicates the post and beam connections in a building several stories above that given in Fig. 41b. The lowest column is provided with a suitable base plate, which not only affords a square bearing, but protects the timber from the water used in cleaning the floor, and which would otherwise start decay in the joint.

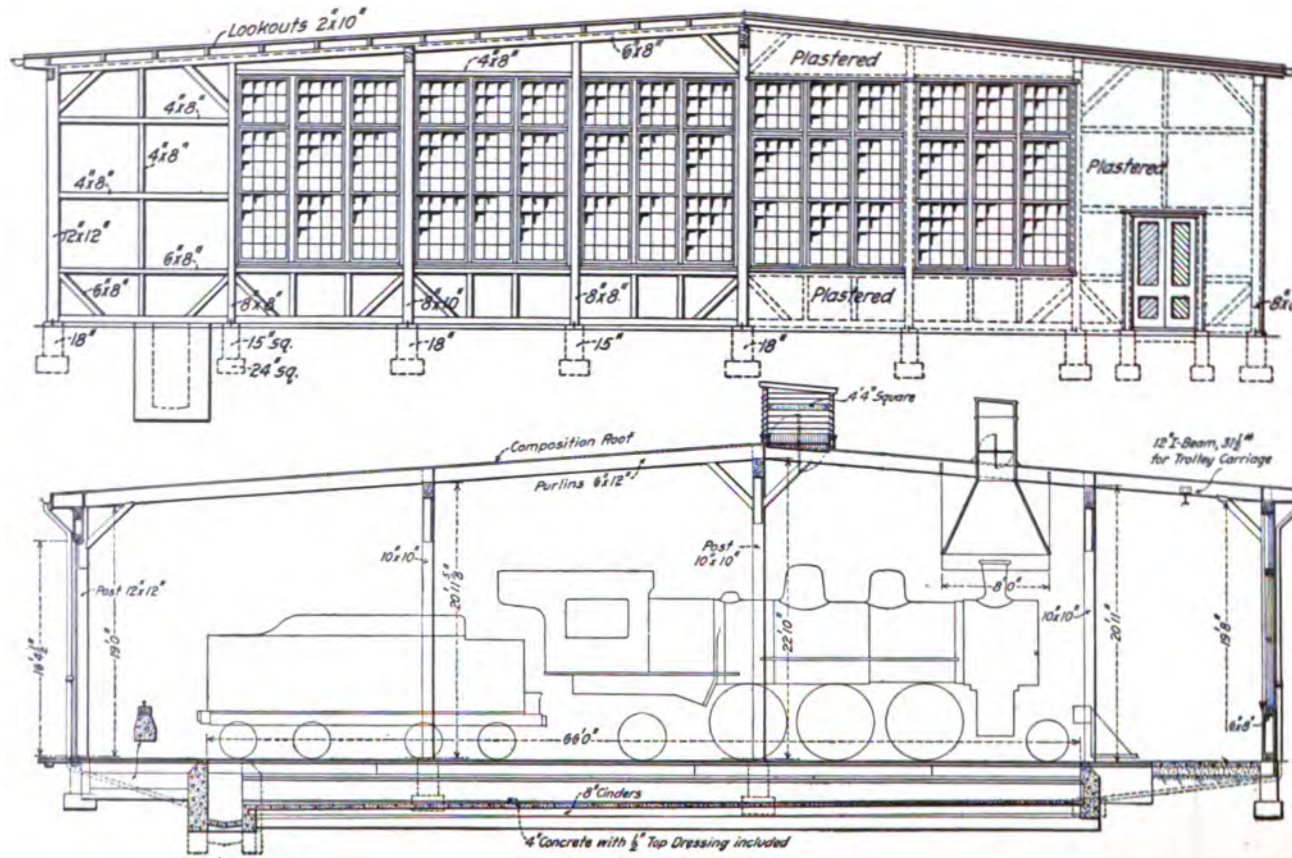
Fig. 47f shows the design of a freight car in which ample provision is made for the bearings against the side of the fibers of the top and bottom chords of the side trusses, both for the diagonals and verticals. Cast-iron bearing caps are sometimes used in wooden trestle construction, as indicated in Figs. 52f, g, and h. A dowel is cast on the cap to hold the upper timber in position. The caps increase the life of the structure by protecting the joints from holding water. See Railroad Gazette, vol. 23, page 260, April 17, 1891.



FIGS. 52f, g, and h. Cast-iron Bearing Caps.

ART. 53. ANGLE BRACES.

An angle brace is a diagonal strut or tie connecting the vertical and horizontal members of a frame, in order to prevent any change in the angle between them, beyond a small elastic defor-



R.R. Age Gaz.

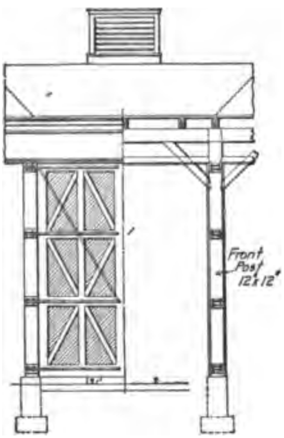
FIGS. 53*a* and *b*. Elevation of End Walls and Cross-section of Round House at Decatur, Ill., Wabash Railroad.

mation. Beams are frequently connected to their supporting columns by an angle brace on each side of the latter. Illustrations of their use are given in Arts. 67 (Plates III and IV), 70, and 76, as well as in this article.

The principal function of braces is to resist compression, and for this purpose the toe at each end engages a notch in the adjacent member, forming a step joint (Fig. 53*c*). Frequently, however, a mortise and tenon is added, and a treenail connects the two pieces to hold them rigidly together. A number of excellent illustrations of the use of angle braces are given on the inset accompanying an article on the Intramural Elevated Electric Railway of the World's Columbian Exposition, in Engineering News, vol. 28, page 362, Oct. 20, 1892. The braces at the feet of the posts are bolted to the posts and sills. Cast-iron braces and brackets are used in some cases to stiffen the top of the bent. In Fig. 53*e* the angle braces are framed into bolsters, which are keyed to the sloping beams supporting the roof.

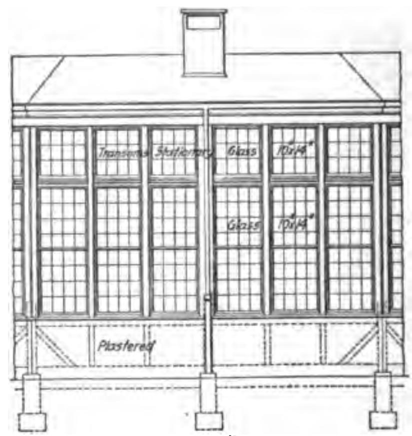
Fig. 55*a* shows the framing of the end wall of a round house, in which an unusually large percentage of the area is glazed. The balance is covered with a heavy coating of cement plaster on expanded metal, which forms the finish of the wall both inside and out. The timber frame is braced by means of short diagonal braces, as indicated on the diagram, the left side of which shows the framing and the right side the finish. The posts are held in position on the concrete footings by dowels. In Fig. 53*b* the braces connecting the posts to the roof timbers, both longitudinally and transversely, may be seen. The inner wall has solid doors with diagonal battens (see Fig. 53*c*), and the short space between them and the roof is likewise finished with plaster. For the description of other features of the building, see Railroad Age Gazette, vol. 45, page 1298, Nov. 6, 1908, and Journal Association of Engineering Societies, vol. 41, page 277, Dec., 1908.

A series of comparative tests of bracing for wooden bents was made in 1897 at the Michigan College of Mines, the results of which were presented to the Lake Superior Mining Institute, in a paper by EDGAR KIDWELL, and published in vol. 4 of the Proceedings. Two forms of wooden angle braces were used,



Front Elevation.

FIG. 53*c*.



Rear Elevation.

FIG. 53*d*.

one with a single step joint accurately framed to a driving fit, the brace being fastened with wire nails; and the other without the step, but connected with mortise, tenon, and treenail. Another bent had full diagonal braces; three others had cast-iron brackets or knees with different details, such as are in use in the mining regions; while cast-iron braces of T-section, and with bearing plates and lugs at the ends, were used with two methods of fastening.

The tests were made both for strength and stiffness. The results indicate that "when new and under conditions in average practice, the wooden braces are fully equal in rigidity and strength to any form of iron braces now in use. The strength of a structure in which wooden braces are used will not be much affected when the braces shrink, since a slight distortion

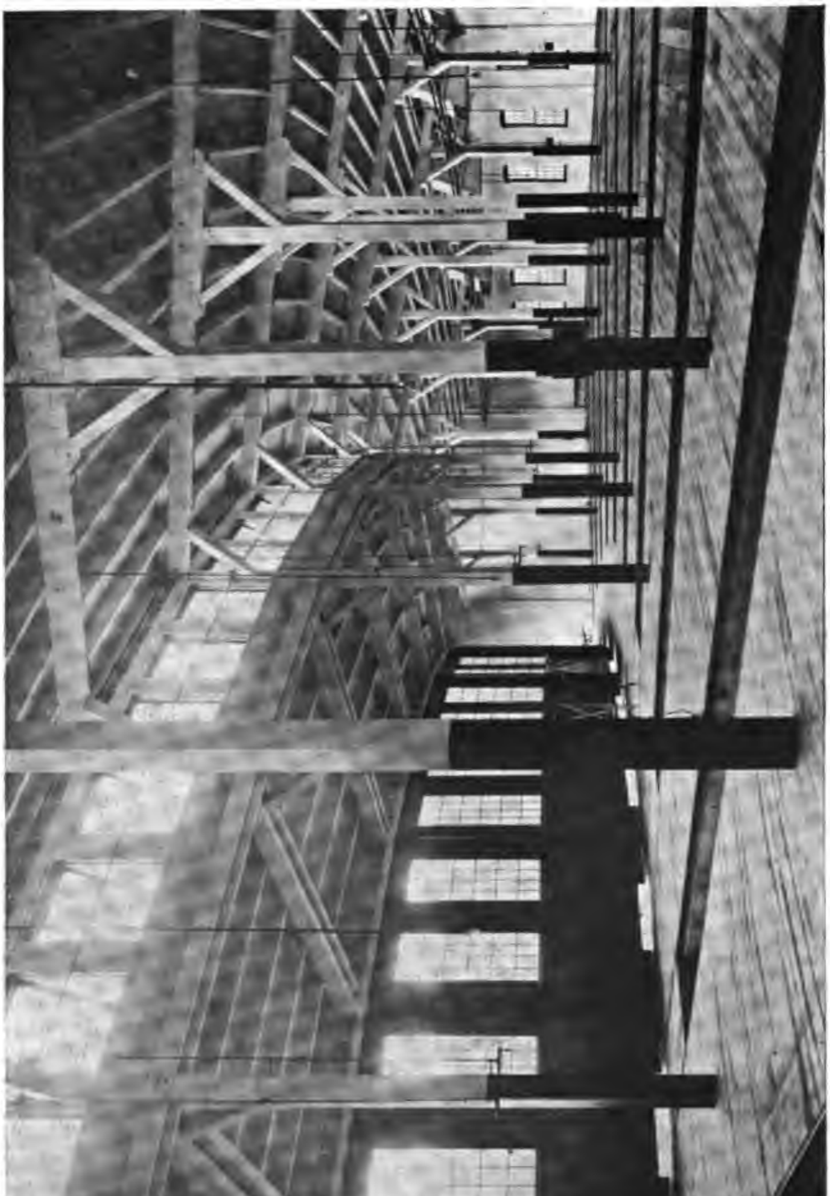


FIG. 53c. Round House of the Ontario & Western Railroad at Middletown, N.Y.

of the building will bring them into play again; but the shrinkage will certainly destroy the rigidity of the structure, and it is this fact which has caused some of the mines of Lake Superior to abandon the wooden braces for iron ones. . . . The experiments seem to indicate that for deflections within a reasonable limit, the bents with wooden braces are the better, and if restrained or loaded on top, they will exceed in rigidity, and probably in strength, the bents braced with iron knees of the usual pattern. . . . No form of iron knees tested gave a very rigid bent, as the latter had to be deflected more than could be allowed in practice before the bents exhibited much strength. If iron knees are used, it is better to use the plain pattern, instead of the usual form provided with lugs or cogs. The cast-iron brace gives a more rigid structure than any of the iron knees, and when merely spiked on is strong enough for any structure not subject to heavy shocks." KIDWELL's paper gives full details of loads and deflections, half-tone views of the bents upon completion of the tests, and diagrams showing the relative deflections of the bents.

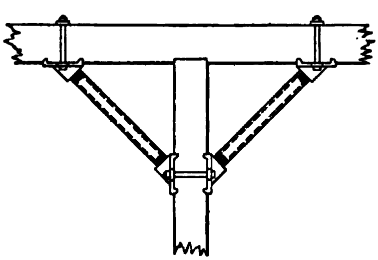


FIG. 53*f*. Kidwell Angle Brace.

Fig. 53*f* shows an adjustable type of angle brace, consisting of cast-iron angle blocks, with lugs to engage notches in post and beam, and a pipe with right and left threads at the ends to screw into the angle blocks.

CHAPTER IV.

WOODEN ROOF TRUSSES.

ART. 54. TYPES OF ROOF TRUSSES.

A 'truss' consists of a frame in which each member is subject only to tension or compression. To avoid flexure in the members their axes should intersect in a point at each joint, and the loads should be applied only at the joints, which are called 'panel points.' Geometrically a truss is composed of a series of triangles. Theoretically the members of a truss are free to turn at the joints, but practically the joints are more or less rigid, due to the methods of construction employed.

The upper line of members from one support to the other forms the 'upper chord,' and the lower line of members the 'lower chord.' The members connecting the chords are called 'web members,' or 'braces,' some of which take compression and are known as 'struts,' while others take tension and are called 'ties.' A 'simple truss' is one having only two supports, and under vertical loads has vertical reactions.

A roof truss is designed especially to support a roof, but it may also support a ceiling, and, in special cases, may support one or more floors below by means of rods, when it is necessary to avoid all columns in any given story. In ordinary construction the roof covering and the laths or sheathing underneath it are supported by 'rafters' which are parallel to the slope of the roof surface; the rafters rest on 'purlins,' which are horizontal beams extending from one truss to another at the panel points of the latter; and the trusses are supported on the wall plates of the outer walls of the building. In special cases one or

more of these elements may be omitted or replaced, as, for example, when the purlins are spaced close together and support the roof covering directly, without the aid of rafters, or when a heavy sheathing of planks is used to support the covering of slate or tile, and rests directly on the trusses. When the purlins rest on the upper chord of a truss between panel points, the chord members are subject to flexure or bending in addition to compression, and the truss then fails to conform to the definition of a true truss.

A wooden roof truss is built principally of wood, but may have iron or steel rods for its tension web members. When the lower chord of a truss is also constructed of steel, it is generally known as a 'combination truss.' Various kinds of fastenings are used at the joints; the timbers composing the chords usually continue past the intermediate joints and are spliced between panel points; while the web struts are connected to the chords by many of the forms of joints or fastenings described in Chapters I and II. In wooden trusses the tension rods pass through the chord timbers and take bearing on washers, while in combination trusses pins are used to connect the members meeting at those joints respectively, where two or more iron or steel members come together. See illustrations in Arts. 67, 69, and 70.

Figs. 54*a* to *h* give the skeleton diagrams of the principal types of wooden roof trusses. In Fig. 54*a* the truss has ver-

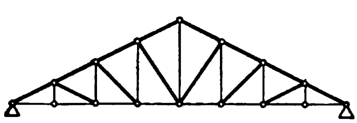


FIG. 54*a*. English Roof Truss.

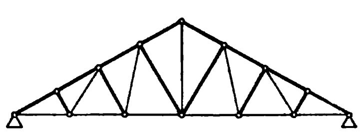


FIG. 54*b*. Belgian Roof Truss.

tical tension rods and diagonal wooden struts. This type is more extensively used than any other. The number of panels depends upon the span, and when there are only two panels it becomes the king-post truss. When the king-post truss was

built entirely in wood, the vertical tension member was called the 'king-post,' but at present a rod is nearly always used instead.

In Fig. 54*b* the wooden struts are perpendicular to the upper chord and the diagonal tie-rods slope upward toward the center. The struts are shorter than in the preceding form, provided the rise is the same. The number of panels in the lower chord is two less than in the upper chord. This type is not adopted as frequently as its merits deserve. When the four middle panels are omitted, this form reduces to that of the combination truss in Fig. 70*c*.

When it is desired to construct the Belgian truss with the longest struts both meeting the lower end of the vertical center rod, it is necessary to have the rise of the truss conform to this requirement. For a truss of 6 panels the rise must be $l/\sqrt{8}$ or $0.3536\,l$, in which l is the effective span. For 8 panels the rise is $l/\sqrt{12}$ or $0.28867\,l$; and for 10 panels, the rise is $l/\sqrt{16}$ or $0.25\,l$. There is frequently no objection, however, to using a lower and more economical rise than that of $0.3536\,l$, by using 2 inclined rods diverging from the peak of the truss.

Figs. 54*c*, *d*, and *e* give forms suitable for very short spans, the first two requiring counter braces in the middle panel as in-

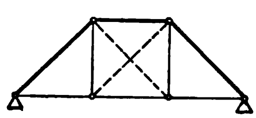


FIG. 54*c*.

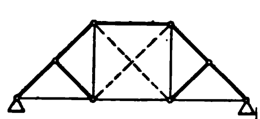


FIG. 54*d*.

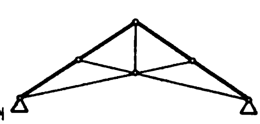


FIG. 54*e*.

dicated by broken lines, to resist wind pressure. Fig. 54*e* is called a scissors truss. The prolongation of the lower chord members past the center should be indicated as compression members in the diagram. This form is often used for roofs having a steep slope.

Fig. 54*g* is well adapted for construction in wood throughout. The long diagonal tension members serve to stay the longer

struts by being connected at their intersection; but such connection does not interfere with the stresses in either. The dotted

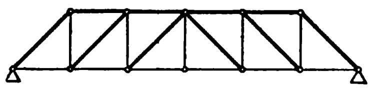


FIG. 54*f*. Howe Truss without Counters.

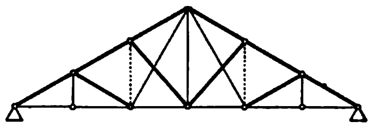


FIG. 54*g*.

lines do not represent members, but merely the fact that the two panel points are in the same vertical.

Figs. 54*f* and *h* are trusses with horizontal chords, and are used either to support flat roofs or to serve the purpose of trussed purlins for a sloping roof. Both of these types are also used for bridge trusses, but the former then requires counter braces on account of the moving load. The details of the Howe truss are very simple. The lattice truss is built in wood throughout and all of its members may be composed of planks which are connected at the intersections with wooden pins, and occasional bolts to keep the planks in contact.

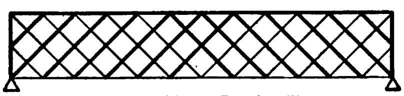


FIG. 54*h*. Town Lattice Truss.

Sometimes Fig. 54*f* is modified slightly by inclining the upper chord parallel to the slope of a roof as in Fig. 69*a*. The upper truss on Plate V has an inclined upper chord, but the web members form a series of triangles with varying heights, their corresponding sides toward the middle being parallel to each other. A similar form is given in Fig. 70*b*. A modification of the form of Fig. 70*c*, which is adapted to six panels, is given in the combination truss on Plate V.

For spans of about 30 feet or less the roof covering may be supported by inclined beams and purlins as on the left side of the freight station shown on Plate V. On the right side the beams are lighter in section, and are reënforced by a straining beam and sloping struts, while the purlins are small in section

and closely spaced. For spans exceeding 30 feet it is usually necessary to provide either intermediate supports, as by a line of columns, or to employ trusses.

The economical limits of span between which wooden roof trusses of various types may be used, as determined by the experience of architects and builders, are given in Chaps. I and III of Trussed Roofs and Roof Trusses, by F. E. KIDDER, or in Architects' and Builders' Pocketbook, by the same author.

Prob. 54*a*. Consult the references in the preceding paragraph and prepare a table containing the economical limits of span for the different types of wooden roof trusses.

Prob. 54*b*. Read the article on Some Celebrated Timber Roofs, by T. ROGER SMITH, in American Architect, vol. 17, pages 259 and 279, May 30 and June 13, 1885.

ART. 55. WEIGHTS OF ROOF TRUSSES.

The loads which are to be supported by a roof truss consist of the dead load, which includes the weight of the truss itself and of the roof covering and other fixed loads supported by it, the snow load, and the wind load. The snow and wind loads are usually specified, or must be determined by the designer in accordance with the climatic conditions of the locality where the building is to be erected. Sometimes a building is in a sheltered location that may permit the wind load to be reduced; but frequently the effect of high winds is not given sufficient consideration in the design of roofs and their supporting framework. The general values employed by engineers are given in Roofs and Bridges, Part I, Art. 2, and in Part II, Arts. 16 and 19.

The most complete list of weights for all the different roof coverings in use in this country, and tables of approximate weights of rafters, purlins, and roof trusses are given in the handbook to which reference was made in the last article.

By comparing the classified weights of 121 designs of roof trusses of spans ranging from 48 to 96 feet, with a rise of one-

sixth to one-third of the span, spacing of trusses from 8 to 12 feet, a snow load of 25 pounds per square foot, and a normal wind load corresponding to a pressure of 40 pounds per vertical square foot, the following formula was deduced for the weight of wooden roof trusses of the types indicated in Figs. 54*a* and *b*:

$$W = \tfrac{1}{2} al (1 + 0.15\, l) \qquad (1)$$

in which W is the weight of one truss in pounds, a the distance between centers of trusses, and l the span, both a and l being expressed in feet.

The designs were made in 1908 for longleaf yellow pine and Western hemlock, all sizes of timbers being in full inches, whether odd or even, thereby securing a fairer basis for comparison. It was found that the trusses of the latter wood are about two percent heavier than those of the former, but that is chiefly due to using relatively lower unit-stresses than those given in Art. 82. The average weights of the trusses for the two types used, namely the English and Belgian (Figs. 54*a* and *b*), are practically the same. A rise of one-fourth of the span gives the lightest truss. For spans below 72 feet, the 6-panel trusses are lighter than those with 8 panels, while for a span of 72 feet the difference is very slight, thus indicating that 12 feet is about the economical panel length. The weight of cast-iron and steel in the trusses varies from 12 to 20 percent of the total weight, the average being 15.3 percent. The percentage diminishes as the span increases. If 10 percent of the lowest and 10 percent of the highest values be omitted, the percentages range from 13 to 18.

Since the entire weight of the members, including the details, cannot be computed with precision until the details are designed and the drawing made, it is desirable to compare the total weight of the truss with what may be termed the theoretic weight, which is obtained by means of the adopted gross section areas

of the members and their lengths from center to center of panel points. The percentage to be added to the theoretic weight to obtain the actual weight was found from an analysis of 97 designs to vary from 13 to 40 percent, the average being 22.4 percent. The percentage also diminishes as the span increases, but does not appear to depend upon the ratio of rise to span. If 10 percent both of the lowest and of the highest values be omitted, the percentage ranges from 14 to 33. The effect of using unit-stresses of three-quarters of the magnitude of those for longleaf yellow pine is to increase the average percentage to be added to the theoretic weight from 18.2 percent to 25.6 percent. It must be remembered that the unit-stresses for iron and steel remain unchanged, while a large proportion of the weight of details to be added to the theoretic weight consists of cast-iron and steel details.

An investigation was made by N. CLIFFORD RICKER for the purpose of obtaining a more accurate formula for the weight of wooden roof trusses than those in existence at the time, the results of which are published in Bulletin No. 16 of the University of Illinois Engineering Experiment Station, August, 1907. The type of truss chosen is Fig. 54*a*, and the kind of wood is longleaf yellow pine. The general arrangement taken was to have a purlin at each panel point of the truss, which supported rafters on which was laid $\frac{7}{8}$-inch sheathing, covered by painted tin. The snow load was assumed at 20 pounds per horizontal square foot, and a normal wind load corresponding to 30 pounds per vertical square foot.

Ten trusses were designed for spans of 20 to 200 feet, varying by 20 feet, with a rise of one-fourth of the span. The panel length was taken 10 feet in all cases, and the spacing between trusses 20 feet. The following formula was deduced from the computed weights of these designs:

$$w = \frac{l}{25} + \frac{l^2}{6000}, \tag{2}$$

in which w is the weight of the truss in pounds per square foot of horizontal projection of the roof supported, and l the span in feet. A similar series of designs was made by using white pine instead of longleaf yellow pine, and the trusses were found to be somewhat lighter.

A comparison of five designs of 200-foot trusses, with a rise of 50 feet, and spaced 10, 15, 20, 25, and 30 feet respectively, shows that the total weight of the roof is a minimum for the spacing of 15 feet. A comparison of trusses of the same span, and spaced 20 feet, but with 8, 10, 12, 14, 16, 18, and 20 panels, respectively, gives a minimum total weight of roof for panels 20 feet long. Another series of trusses of the same span and spacing, but with rises of 20, 25, 30, 35, 40, 45, and 50 feet respectively, indicate that the total weight of roof is a minimum for a rise of 35 feet, which is a little over one-sixth of the span, the angle of inclination being nearly 20 degrees.

Still another series of trusses of the same span and spacing was designed, with from one to five purlins for each panel length, subjecting the upper chord to combined flexure and compression. The results show that the weight of the roof covering, sheathing, and rafters combined decrease as the number of purlins is increased; the weight of the purlins increases, the additional weight of the connections slightly diminishes, and the weight of the truss increases. Considering the total weight of roof, it was found that no advantage results from the use of more than 2 purlins per panel 25 feet long, or of more than one for panels of ordinary length. Additional designs show that it is not economical to raise or camber the lower chord, indicating that this is done only for the sake of appearance.

Prob. 55. A roof truss has a span of 80 feet, and the spacing is 14 feet. Compute the weight of the truss by formulas (1) and (2) and find the percentage of difference between the results. Make a similar computation for a span of 100 feet, and a spacing of 12 feet.

ART. 56. STRESSES IN RAFTERS.

A common rafter which supports roof covering is preferably regarded as a simple beam under uniform load, the direction of which is such that one component is parallel to the rafter and the other perpendicular to it. In Fig. 56*b*, the vertical arrows represent the weight of roof covering, sheathing, and the rafter

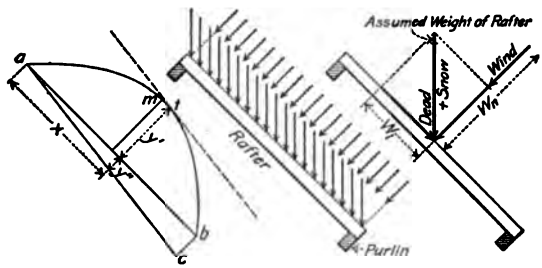


FIG. 56*a*. FIG. 56*b*. FIG. 56*c*.

itself, as well as of the snow load, while the arrows perpendicular to the rafter represent the wind load. In Fig. 56*c*, the decomposition of the total load into the longitudinal and normal components W_l and W_n is indicated, it being remembered, however, that the loads are distributed, and not concentrated.

In Fig. 56*a*, the parabola *amb* gives the bending moment diagram for the rafter due to the normal load, but it is drawn to such a scale that the ordinates represent the unit-stresses in the outer fiber due to flexure. The ordinate at the middle equals $6\,W_n l'/8\ bd^2$, in which l' is the inclined span of the rafter; b and d the breadth and depth of its cross-section.

If the rafter be regarded as fastened to the purlin at the lower end only, then the unit compressive stress due to the longitudinal component of the load varies as the ordinates of the triangle *abc* in which *bc* equals W_l/bd. The maximum unit-stress is therefore represented by the maximum ordinate $y = y' + y''$, which is located at the point of contact t of a parallel to *ac* drawn

tangent to the parabola. By mechanics, the section is located at a distance from the upper support equal to $x = \frac{1}{2} l' + \frac{1}{6} d \tan\alpha$ in which α equals the angle of inclination of the rafter to the horizontal. If, however, the rafter be regarded as fastened only at the upper end, the maximum unit-stress in the outer fiber is tension, and the section in which it is located is the same as before. This shows that the maximum stress is so near the middle that practically its value may be taken the same as that at the middle of the span.

For example, let $l = 10$ feet $= 120$ inches, $\alpha = 60$ degrees, and let the depth of the rafter be assumed as 6 inches. Then

$$x = \tfrac{1}{2}(120 + \tfrac{1}{6}/6 \times 1.73) = 61.75 \text{ inches},$$

or only 1.75 inches below the middle of the span. The total vertical load given, including slate, sheathing, rafter, and snow, is 562.5 pounds, and the normal wind load is 800 pounds. The longitudinal component is $562.5 \times 0.866 = 487$, and the total normal component is $800 + 562.5 \times 0.5 = 1081$ pounds. The maximum bending moment is $1081 \times 120/8 = 16\,215$ pound-inches. For an allowable unit-stress in the outer fiber of 1800 pounds per square inch, the resisting moment of the rafter is

$$1800 \times b \times 6^2/6 = 10\,800 b.$$

Equating these moments, there is found $b = 1.50$ inches. Taking a width of 2 inches, the direct compression [or tension] is found to be $\frac{1}{2}(487)/(2 \times 6) = 20$; and that due to flexure is

$$16\,215 \times 6/(2 \times 6^2) = 1351;$$

making the total unit-stress 1371 pounds per square inch. These results show that the unit-stress due to the longitudinal component of the load is so small in comparison with that due to the normal component, that it may be neglected in the design of rafters. A rafter is seldom given as steep a slope as 60 degrees, so that generally the direct stress is relatively smaller.

Prob. 56*a*. Ascertain whether a 2 by 5-inch rafter may be used in the example given above, without exceeding the allowable unit-stress of 1800 pounds per square inch.

Prob. 56*b*. A 2 by 6-inch rafter on a roof truss having a rise of one-fourth of the span is supported by purlins spaced 12.02 feet center to center. Its total vertical load, including its own weight, is 670 pounds, and the normal wind load is 572 pounds. Compute the maximum unit-stress in the rafter.

ART. 57. STRESSES IN PURLINS.

When common rafters are employed, a purlin is practically a simple beam supporting concentrated loads at regular intervals, each of these loads being the resultant of the normal and vertical loads on one rafter, including the weight of the latter. The purlins are frequently placed with their longer sides vertical; and while this direction gives the largest resistance of the purlin to the vertical loads, it may be quite unfavorable for the wind loads, the maximum stress being produced when both vertical and normal loads act together. It is possible for this arrangement of the purlin to render it weaker than if it be placed with its longer sides parallel to any other direction. This condition occurs when the neutral axis coincides with the diagonal of the cross-section.

The purlin will have its largest resisting moment when the longer sides are placed parallel to the direction of the resultant of its maximum loads. When beam hangers are employed to support the purlin from the upper chords of the trusses, this position should be used, for then the cost will not be greater than for any other position.

For ordinary roof trusses in which the rise does not exceed about one-third of the span, it is customary to rest the purlins on the upper chord, thus making the longer sides of any purlin perpendicular to the slope of the rafter, and avoiding the necessity of notching the purlin over the chord of the truss. It may frequently be possible, however, that the cost of the notching is less than the amount of lumber saved in the purlins.

According to the principles of mechanics, the neutral axis of a beam is not perpendicular to the direction of the load when the load is not parallel to any side of a beam of rectangular cross-section. Fig. 57*a* represents the section of a purlin in which the resultant load makes an angle α with the longer sides, and the angle between the neutral axis and the shorter sides is designated by β. By mechanics, the formula for the angle of inclination β of the neutral axis is

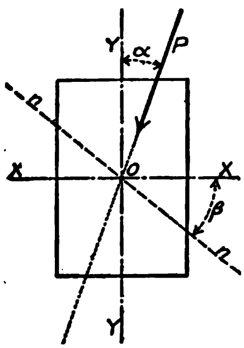


FIG. 57*a*.

$$\tan\beta = (d^2/b^2)\tan\alpha, \tag{1}$$

in which b and d are the breadth and depth of the cross-section respectively.

Representing the moments of inertia about the x- and y-axis by I_x and I_y respectively, the moment of inertia about the neutral axis is

$$I = I_x \cos^2\beta + I_y \sin^2\beta. \tag{2}$$

For rectangular sections this formula may be put into a more convenient shape as follows (see handbook Cambria):

$$I = \tfrac{1}{12}bd[(d\cos\beta)^2 + (b\sin\alpha)^2], \tag{3}$$

the values $d\cos\beta$ and $b\sin\beta$ being most conveniently obtained by graphics. The distance c from the neutral axis to the outer fiber is

$$c = \tfrac{1}{2}(d\cos\beta + b\sin\beta). \tag{4}$$

For example, let it be required to design a purlin having a span of 12 feet, and supporting rafters 2 feet apart, the normal component of the load on each rafter being 1140 pounds, and the longitudinal component 304 pounds. The resultant load is 1180 pounds, and the maximum bending moment occurs in the purlin when a rafter is placed at its center.

Let an approximate design be made first upon the assumption

that the longitudinal components of the loads on the rafters are resisted entirely by the sheathing. The maximum bending moment then is $M' = \frac{1}{2}(1140 \times 5)6 - 1140(4+2) = 10\,260$ pound-feet $= 123\,120$ pound-inches. Assuming a section of 6 by 9 inches, the weight of the purlin is 162 pounds, at 3 pounds per foot board measure. The bending moment due to its own weight is $M'' = \frac{1}{8} \times 162 \times 12 \times 12 = 2915$ pound-inches, making $M = M' + M'' = 126\,040$ pound-inches. For an allowable unit-stress of 1650 pounds per square inch, the resisting moment is $1650\,bd^2/6$, whence $bd^2 = 458.3$. If the depth $d = 9$ inches, the breadth is $b = 5.66$ or 6 inches.

Let the section next be assumed as 7 by 10 inches, and its maximum unit-stress computed by the more exact method. The revised weight of the purlin is 210 pounds, $M' = 127\,440$ pound-inches, $M'' = 3780$ pound-inches, and $M = 131\,220$ pound-inches. By equation (1), $\tan\beta = (100/49)(304/1140) = 0.544$, whence $\beta = 28°\ 33'$; $\cos\beta = 0.8784$ and $\sin\beta = 0.4779$; by equation (3), $I = 515.7$ inches^4; and by equation (4), $c = 6.06$ inches. Substituting these values in the general formula for the resisting moment, $S = Mc/I$, there is found $S = 131\,220 \times 6.06/515.7 = 1542$ pounds per square inch. On investigating the 6 by 9 section, the maximum unit-stress is found to be 2270 pounds per square inch, and for a 6 by 10-inch section it also exceeds the allowable limit specified. The section to be adopted is therefore 7 by 10 inches. Since the ratio of the depth to the span of the purlin is less than 150/1650, the horizontal shear need not be considered in its design (Art. 35).

If the construction is so made that the longitudinal component of the load in the rafter is effectively resisted by the sheathing, or if the purlins be placed with their long sides parallel to the resultant load, the 6 by 9-inch section has sufficient strength.

Prob. 57. Find the value of the maximum unit-stress for a 6 by 10-inch section of the purlin under the loads and conditions given in the preceding example.

ART. 58. DESIGN OF A ROOF TRUSS.

The span and loading of a roof truss to be designed is generally given in the specifications, as well as the working unit-stresses to be used for the different kinds of material which are employed in its construction. Sometimes also the type of truss preferred may be specified, and conditions imposed which control the spacing, kind of roofing material, etc. For the purpose of encouraging in students the habit of systematic arrangement in computations, of facilitating reference to preceding portions of the design, and of simplifying the examination of the reports, the following order of design is suggested:

ORDER OF DESIGN.

1. Rafters. Length of upper chord of truss on one side. Inclined span of rafter, or distance between purlins. Roof area supported by one rafter for this span, in terms of spacing x. Weight of sheathing, roof covering, snow, and estimated weight of rafter for this area. Combined dead and snow load. Angle of inclination of roof surface. Normal wind load per square foot, and for the given area. Total normal component of loads. Longitudinal component. Resisting moment for rafters of 2 by 4, 2 by 5, 2 by 6, 2 by 7, and 2 by 8 inches. Corrected weight of rafter.

2. Purlins. Span. Revised normal, longitudinal, and resultant load transmitted by each rafter. Diagram showing position and magnitude of concentrated loads on purlin. Estimated weight of purlin. Maximum bending moment. Required value of bd^2 when sides of purlin are parallel to resultant load. Preliminary assumed cross-section. Revised maximum bending moment. Tanβ, cosβ, and sinβ. Diagram of cross-section, direction of load and of neutral axis. Moment of inertia. Distance to neutral axis from outer fiber. Maximum unit-stress in outer fiber. Revision, if necessary. Final weight of purlin.

3. Estimated weight of truss. Weight per panel. Weights of roof covering, sheathing, snow, and rafters for one panel load determined directly from rafter load. Weight of one purlin. Total dead panel load on upper chord. Snow panel load. Dead panel load on lower chord due to ceiling, if any. Wind panel load. [All panel loads to be expressed in multiples of 10 pounds.]

4. Computation of exact lengths of truss members, center to center of panel points. Diagrams showing these dimensions, all panel loads, combined dead and snow load reactions, and wind load reactions.

5. Truss diagrams drawn to as large a scale as prescribed sheet will allow. See plate posted for general arrangement and notation. Drawing of load line, all panel loads and reactions being laid off in regular order while passing completely around the truss in a clockwise direction. Find dead and snow load stresses by a single diagram, using as large a scale as practicable. Construct wind stress diagram to a larger scale. Mark stresses [expressed in multiples of 10 pounds] on left half of truss diagram in three lines: 1, dead and snow; 2, wind; 3, maximum.

6. Design of sections of truss members. Arrange results in tabular form with the following column headings: Member, length, maximum stress, size, ratio l/d, P/A, section area required, section area furnished, and remarks. Place on the same page a skeleton truss diagram with panel points designated by letters. When upset screw ends are used, give diameter in column of remarks. Leave dimensions of cross-section and section area furnished by lower chord blank until end joint is designed.

7. Skeleton truss diagram showing stresses in web members and their normal and longitudinal components with regard to the upper and lower chords respectively. Also vertical and horizontal component of total stress in end upper chord member, and the horizontal component of the stress at the peak.

8. Diagram showing allowable unit-stresses for compression on section or surfaces inclined to the wooden fibers, drawn to a scale of 400 pounds to an inch.

9. Design of washers, arranged in regular order for various panel points. For ogee or plate washers: net bearing area; diameter of bolt; diameter of hole; gross area of base; diameter of round, or side of square washer; net area furnished. For beveled washers: longitudinal and normal components of stress in rod; bearing areas of end and base of washer; depth of notch, width, length; long diameter of nut; detail pencil drawing to scale with dimensions, indicating intersection of the three resultant pressures.

10. Other details of intermediate joints, arranged in regular order for various panel points. Angle between section and direction of fibers. Allowable unit compression. Bearing area of strut. Depth of notch or indent. Size and location of dowels. Pencil drawing, with dimensions.

11. Design of end joint of truss. Assumed diameter of inclined bolts to keep parts in position. Size of washers. Bearing surface and depth of toe of upper chord. Bearing on horizontal plate. Bearing surface, number, and depth of lugs on forged shoe. Thickness of lugs. Clear distance between lugs, and from outer lug to end of lower chord timber. Required net section of lower chord. Net and gross depth. Design of key between chord and bolster. Bearing area and depth of notch. Length required to resist shear. Length required to resist moment of rotation of key. Size of bolt and washers to resist vertical reactions of key. Clear distance between inner lug and key. Detail pencil drawing to scale, with dimensions.

12. Connection of purlins to upper chord of truss. Number and size of nails to resist component of resultant load parallel to upper chord. Investigate bearing surfaces of purlins. Other details, if any.

13. Design of anchorage. Width of wall plate. Depth of notch over wall plate. Connection to resist lifting of truss by internal wind pressure. Anchor bolts.

14. Design of splices. Splice in upper chord. Splices in lower chord.

15. Detail drawing, with complete dimensions required for construction. See Plates I-IV, blue prints, or other drawings exhibited, showing arrangement, dimensions, lettering, and execution.

16. Bill of material, properly classified, for one complete bay of roof.

17. Estimate of cost for one complete bay of the roof.

18. Analysis of weight of one roof truss. Weight of lumber. Weight of steel. Weight of cast-iron. Weight of metal expressed as percentage of total weight. Theoretic weight. Ratio of computed or final weight to theoretic weight. Excess or deficiency of estimated weight of truss with respect to final weight.

ART. 59. SPECIFICATIONS, RAFTERS, AND PURLINS.

SPECIFICATIONS.

Let it be required to design a wooden roof truss of the type illustrated in Fig. 54*a*, with a span of 60 feet, a rise of one-fourth of the span and 6 panels, the spacing between centers of trusses being 12 feet. The truss is to be designed for construction of Western hemlock with vertical tie-rods of soft structural steel, forged shoe plates of medium steel, and with washers of cast-iron or steel. The roof covering is to consist of slates, $\frac{3}{16}$ inch thick, laid on one-inch sheathing, supported by rafters spaced not less than 16 nor more than 24 inches apart, the rafters being carried by purlins placed at, or very near to, the panel points of the trusses. The sheathing, rafters, and purlins are also to be of

Western hemlock, which is to be assumed to weigh 36 pounds per cubic foot.

The snow load is to be 20 pounds per square foot of horizontal projection, and the normal wind load is to be computed from HUTTON's formula for a pressure of 40 pounds per vertical square foot. The truss is also to be designed for the future addition of a ceiling estimated to weigh 20 pounds per square foot. In proportioning the different parts of the structure provision is to be made for the maximum stresses due to dead, snow, and wind loads.

The allowable unit-stresses expressed in pounds per square inch are to be as follows:

For Western hemlock. Extreme fiber stress in bending, 1650; modulus of elasticity, 1 480 000; shearing parallel to the fiber, 240; longitudinal shear in beams, 150; compression perpendicular to the fiber, 330; compression parallel to the fiber, 1800; compression for columns under 15 diameters in length, 1350; formulas for longer columns, $1800\,(1 - \frac{1}{60}\,l/d)$. The allowable compression under washers is to be 25 percent greater than when the bearing covers the full width of the member.

For steel tie-rods and bolts in tension, 15 000; for medium steel in flexure, 25 000; for rivets in shear, 10 000; for rivets in bearing, 20 000.

DESIGN OF RAFTERS.

To obtain the span of the rafters it is necessary to compute the length of the upper chord for one-half of the truss span. This is most conveniently done with the aid of a table of squares of the modern type. The half span of the truss is 30 feet and the rise 15 feet; the length of the upper chord is 33 feet $6\frac{1}{2}$ inches, and hence the inclined span of a rafter is 11 feet $2\frac{3}{16}$ inches, or 11.18 feet. The horizontal projection of its span is 10 feet.

Let x be the distance in feet between centers of rafters, then the roof area supported by a rafter is $11.18x$ square feet. Assuming the weight of the slate at 8 pounds, and the approximate weight of rafter at 1.5 pounds per square foot; and the other unit weights as given in the first and second paragraphs of this article, the weights supported by a rafter are:

Slate,	$11.18x \times 8$	=	$89.44x$ pounds
Sheathing,	$11.18x \times 3$	=	$33.54x$ pounds
Rafter,	$11.18x \times 1.5$	=	$16.77x$ pounds
Snow,	$10x \times 20$	=	$200x$ pounds
Total vertical load		=	$339.75x$ pounds

4

The angle of inclination of the roof surface is $26^\circ\ 34'$ since its tangent is 0.5, its cosine is 0.8944 and its sine 0.$\cancel{6}$472. According to Roofs and Bridges, Part II, Art. 19, the normal wind pressure is 23.8 pounds per square foot, and the wind load on the rafter is $11.18x \times 23.8 = 266.1x$ pounds. The normal component of the dead and snow load is $339.75x \times 0.8944 = 303.9x$ pounds, making the total normal load $570x$ pounds. The longitudinal component of the vertical load is $339.75x \times 0.4472 = 152x$ pounds.

The resisting moment of a 2 by 4-inch rafter is $1650 \times 2 \times 4^2/6 = 8800$ pound-inches; that of a 2 by 5 is 13 750; of a 2 by 6 is 19 800; of a 2 by 7 is 26 950; and of a 2 by 8-inch section is 35 200 pound-inches. The bending moment for the normal load is $570x \times 12.18 \times 11/8 = 9559x$ pound-inches. Equating the bending moment with the resisting moments in succession, the spacing of the rafters x is as follows: for 2 by 4-inch section, 0.921 feet or 11.05 inches; for 2 by 5, 17.26 inches; and for 2 by 6, 24.86 inches. In accordance with the specification, either 2 by 5-inch rafters, spaced 17 inches, or 2 by 6-inch rafters spaced 24 inches may be adopted. The latter size will be selected, since the sheathing has ample strength for a spacing of 24 inches.

The revised total load on the rafter may next be computed: slate, 179; sheathing, 67; rafter, 34; snow, 400; or a total vertical load of 680 pounds. The normal load is 532 pounds. The total normal component is 1140 pounds, and the longitudinal component is 304 pounds. The unit-stress in the outer fiber due to flexure is 1592, and that due to direct compression is 13, making the total 1605 pounds per square inch.

Sometimes it becomes necessary to design a roof for stiffness, as well as strength, on account of the type of construction adopted. In that case the rafters are to be designed not to deflect more than say $\frac{1}{360}$th of the span. As an example, the necessary computations will also be given. The formula for the deflection of a simple beam uniformly loaded is $\Delta = 5\ Wl^3/384\ EI$. $I = bd^3/12$ for a beam of rectangular cross-section. The deflection allowed is $11.18 \times 12/360 = 0.373$ inch.

Since experiment shows that for long-continued loading the modulus of elasticity is only about one-half of its value, under an immediate application of the same load, the dead load is doubled and added to the snow and wind loads. The total normal load is $W = [2\,(89.44 + 33.55 + 16.77)x + 200x] \times 0.8944 + 266.1\,x = 695\,x$. Using the same section of rafter as that selected above, or 2 by 6 inches, and substituting in the formula for deflection, there is obtained the equation:

$$5 \times 695\,x\,(11.18 \times 12)^3 = 0.373 \times 384 \times 1\,480\,000 \times 2 \times 6^3/12,$$

the solution of which gives $x = 0.91$ feet or 10.92 inches. This spacing falls below the limit specified. The corresponding spacing for a 2 by 7-inch section is 17.33 inches, and for a 2 by 8-inch section is 25.87 inches. Either the former section with a spacing of 17 inches, or the latter with a spacing of 24 inches, might be adopted. It will be observed that the method of deflection requires 2 inches more depth than for strength.

DESIGN OF PURLINS.

In Art. 57 the design of a purlin is given for the same loads and conditions as those under consideration in this article. On the basis of strength, the 7-by 10-inch purlin will therefore be adopted.

Let the computations, however, be added which are required to design the purlin, so that its deflection shall not exceed $\frac{1}{360}$th of the span, or 0.4 inch. Let it be assumed that the component of the load parallel to the slope of the roof is resisted by the sheathing. It will be sufficiently accurate for this purpose to regard the load as uniformly distributed. The total load normal to the roof surface is 8720 pounds, the dead loads having been doubled. Using the same general formula for deflection as for the rafter, the following equation is obtained:

$$5 \times 8720 (12 \times 12)^3 = 0.4 \times 384 \times 1\,480\,000\, bd^3/12.$$

The value of bd^3 is found to be 6872 inches^3. Assuming $d = 10$ inches, $b = 6.87$ or 7 inches, which gives the same section as that designed for strength, under the assumption that the sheathing does not resist the component of the load parallel to the slope of the roof.

Prob. 59. Assuming that the component of the load parallel to the slope of the roof surface is not resisted by the sheathing, compute the width of the purlin on the basis of deflection, the depth being taken as 10 inches.

ART. 60. TRUSS LOADS AND STRESSES.

For a span of 60 feet and a spacing of 12 feet, the weight of a truss according to formula (1) in Art. 55 is $\frac{1}{2} \times 12 \times 60 (1 + 0.15 \times 60) = 3600$ pounds, and the weight per panel is $3600/6 = 600$ pounds. Since the spacing of the rafters is 2 feet, the number of rafters supported by one purlin, and hence by one panel point of the truss, is $12/2 = 6$. The panel loads consist of the following items:

upper panels. Or, in case the chord is built up of three pieces bolted together, the middle piece may be omitted in the upper panels, and packing blocks inserted between the outer pieces at the panel points, cast-iron separators being used at intermediate points where they are bolted together. Planks are less expensive than large dimension timbers, and have fewer hidden defects in the wood; but as they must be bolted together at close intervals, they require extra cost in bolts and labor which may counterbalance or exceed the initial saving in lumber. Furthermore, since the strength of a composite column is materially less than that of a solid timber of the same gross section, it is not economical generally to use a built-up chord member to resist compression.

The upper chord should be as nearly square in section as possible, as this is the best section for a column. When the dimensions of the cross-section are unequal, it depends upon the quality of the wood whether the long or the short side is to be placed horizontally. Since the entire horizontal component of its stress has to be transmitted into the lower chord, the two chords should have the same width. If the chords are too narrow, the lugs of the end shoes must be deeper, and, therefore, require thicker material. When steel forged shoes are employed, the cost increases rapidly with any decrease in width.

For trusses of ordinary dimensions, it improves the appearance to make all the struts the same width as the chords, giving flush surfaces at the joints. In large trusses it is more economical to reduce the width from the middle toward the ends, but it is not desirable to have too many different sections. Occasionally, the struts are built of two sticks, united by bolts and filler pieces; but they must be designed as composite columns in that case (Art. 50). Sometimes the sections of one or more struts have to be increased on account of the bearing against the side of the upper chord.

When a single tie-rod exceeds $1\frac{1}{2}$ or $1\frac{5}{8}$ inches in diameter, it is usually preferable to substitute two rods of smaller diameter, on account of the distribution of the pressure under the washers.

Prob. 61. In case a steel rod with upset ends were substituted for the plain rod *Dd*, find the diameter of the rod and of its upset ends.

ART. 62. DESIGN OF JOINT DETAILS.

The details at the joints to be designed include the washers, and the depths of notches or indents to resist the longitudinal components of the stresses in the struts, with respect to the chords. It is also necessary to investigate the bearing surfaces of the struts on the sides of the chords, to see whether the section of the strut has to be increased or a bearing block provided.

ALLOWABLE COMPRESSION ON SURFACES INCLINED TO THE FIBERS.

Since most of the bearing surfaces referred to in the preceding paragraph are inclined to the fibers of the wood, it is first required to determine the allowable unit-compression on an inclined surface in terms of the compression on the ends and side of the fibers respectively. Let S' and S'' be the allowable unit-stresses on the ends and side of the fibers respectively, S the allowable unit-stress on a surface which makes an angle θ with the direction of the fibers, P', P'', and P the total compression corresponding to these unit-stresses respectively. Referring to Fig. 62*a*, $dy = ds \sin\theta$, and $dx = ds \cos\theta$. As the resistance is directly proportional to

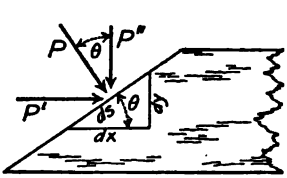


FIG. 62*a*.

the thickness, let the stick in the diagram be taken as one unit in thickness; then $P' = S'dy = S'ds \sin\theta$, and $P'' = S''dx = S''ds \cos\theta$; but $P = P' \sin\theta + P'' \cos\theta$, whence $P = (S' \sin^2\theta$

$+ S'' \cos^2\theta)ds$, and, therefore, the required allowable unit-stress $S = P/ds$ is

$$S = S' \sin^2\theta + S'' \cos^2\theta. \qquad (1)$$

By means of this equation the curve in Fig. 62*b* is constructed, the angles between the bearing surfaces and the direction of the fibers being laid off as abscissas, and the corresponding allowable unit-stresses as ordinates. It will be observed that, for an

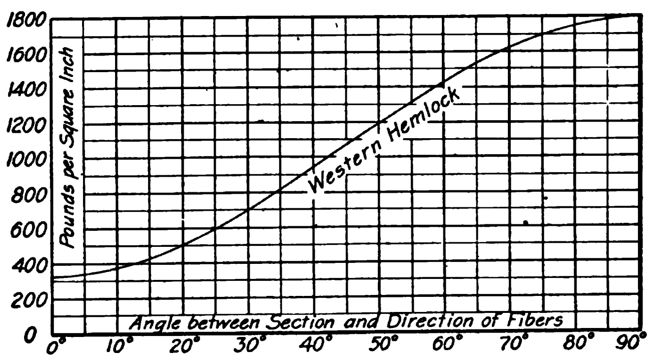


FIG. 62*b*. Compression on Cross-sections inclined to the Fibers.

inclination of 45 degrees, the allowable unit-compression is the mean of S' and S'', which is 1065 pounds per square inch for Western hemlock. The components of the stresses in the web members with respect to the chord members are given on Fig. 62*c*.

DESIGN OF WASHERS.

JOINT *B*. — It will be most convenient to cut a triangular notch into the top of the chord, to receive the bearing of a horizontal washer. In order to cut away as little as possible of the bearing area for the purlin at this panel point, a square steel-plate washer is selected. The angle between the bearing surface and the fibers is 23½ degrees; Fig. 62*b* gives a corresponding unit-stress of 620 pounds, which, if increased 25 percent for washers, becomes 775 pounds per square inch. The net bearing area

required is 2400/775 = 3.10 square inches. The diameter of the bolt is $\frac{5}{8}$ inch, and of the hole in the washer $\frac{11}{16}$ inch; hence the gross area is 3.10 + 0.37 = 3.47 square inches, which requires a washer $1\frac{7}{8}$ inches square. Since this is not a standard size, the 2-inch size is adopted. The thickness according to a manu-

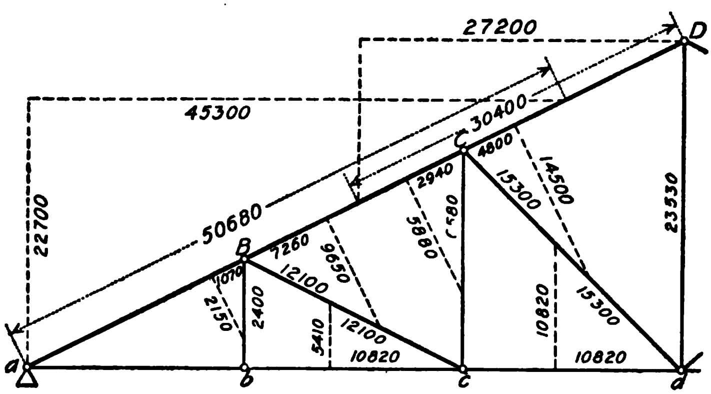


FIG. 62c. Longitudinal and Normal Components of Stresses.

facturer's catalogue is $\frac{3}{16}$ inch. The diameter of square nut required is $1\frac{1}{8}$ inches (see manufacturer's standard sizes in Cambria).

JOINT C.— Net bearing area = 6580/775 = 8.49; diameter of bolt, 1 inch; diameter of hole, $1\frac{1}{16}$ inches; gross area = 8.49 + 0.89 = 9.38 square inches; size of washer required, $3\frac{1}{16}$ inches square; standard size, $3\frac{1}{2}$ inches square, $\frac{5}{8}$ inch thick; diameter of square nut, $1\frac{3}{4}$ inches.

JOINT D.— Net bearing area = 20 530/775 = 26.49; diameter of bolt, $1\frac{5}{8}$ inches; diameter of hole, $1\frac{3}{4}$ inches; gross area = 26.49 + 2.41 = 28.90 square inches; size of washer required, 5 by 6 inches. The diameter of a square nut for the bolt is $3\frac{1}{4}$ inches. The thickness required may be found approximately by assuming that the nut distributes the load over a width of $3\frac{1}{4}$ inches across the 5-inch width of washer, and treating the pro-

jecting length of $1\frac{3}{8}$ inches as a cantilever under uniform load. The result of the computation is 0.571 or $\frac{5}{8}$ inch.

JOINT *d*.— Net bearing area = 20 530/410 = 50.07; diameter of bolt, $1\frac{5}{8}$ inches; diameter of hole, $1\frac{3}{4}$; gross area = 50.07 + 2.41 = 52.48; size of steel washer, $7\frac{1}{4}$ inches square; thickness, $\frac{9}{16}$ inch.

JOINT *c*.— Net bearing area = 6580/410 = 16.04; diameter of bolt, 1 inch; diameter of hole, $1\frac{1}{8}$ inch; gross area = 16.04 + 0.99 = 17.03; diameter of cast-iron ogee washer required, $4\frac{11}{16}$ inches; standard size, 5 inches.

JOINT *b*.— Net bearing area = 2400/410 = 5.85; diameter of bolt, $\frac{5}{8}$ inch; diameter of hole, $\frac{3}{4}$ inch; gross area = 5.85 + 0.44 = 6.29; diameter of washer required, $2\frac{7}{8}$ inches; standard size, 3 inches.

DESIGN OF OTHER JOINT DETAILS.

JOINT *B*.— The longitudinal component of the stress in the 4 by 6-inch strut *Bc* with respect to the upper chord is 7260 pounds. Let it be assumed temporarily that the normal component will act on the larger inclined surface of the indent without slipping, and that the other side is perpendicular to the upper chord. The depth of the indent depends on the allowable bearing for the strut, and not for the chord. The angle between the bearing surface and the fibers of the strut is 37 degrees, and the allowable unit-compression (Fig. 62*b*) is 860 pounds per square inch. The approximate depth of indent needed is, therefore, $7260/(860 \times 6) = 1.41$ inches. Let the sides of the indent be made perpendicular to each other, the shorter one being assumed as $1\frac{1}{4}$ inches. On resolving the stress in the strut into two components respectively perpendicular to the shorter and longer sides of the indent, they are found to be 9500 and 7500 pounds. The angle between the former bearing surface and the fibers of the strut is now changed to 55 degrees and the unit-stress to 1320 pounds per square inch. The required depth is then

9500/(1320 × 6) = 1.20 inches. The assumed depth is therefore adopted. The other bearing surface makes an angle of 15 degrees with the fibers, of the chord, and the allowable unit-com-

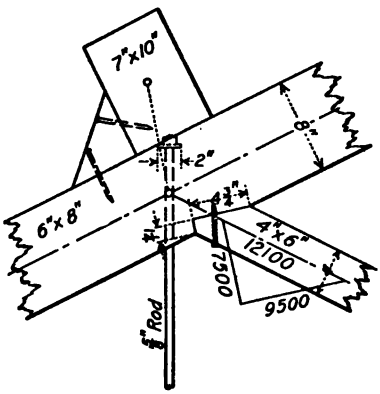


FIG. 62d. Joint *B*.

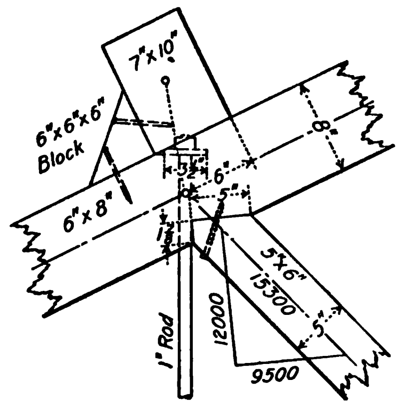


FIG. 62e. Joint *C*.

pression is 430 pounds per square inch. The required width is 7500/(430 × 6) = 4.64 inches. As it measures 4¾ inches, no revision is necessary.

JOINT *C*. — Longitudinal component of stress in 5 by 6-inch strut $Cd = 4800$; angle between assumed bearing surface and fibers of strut = 18½ degrees; allowable unit-stress = 480; approximate depth of indent = 4800/(480 × 6) = 1.67 inches. Assuming 1¾ inches for shorter side of indent, revised component = 9500 pounds; revised angle = 38 degrees; revised unit-stress = 880 pounds per square inch; and required depth of indent = 9500/(880 × 6) = 1.80 inches, which indicates that it should be increased to 1⅞ inches. For the longer side of the indent the component is 12 000 pounds, the angle 20 degrees, the unit-stress 500 pounds per square inch and the required length of the side 4 inches, while 5 inches is provided.

JOINT *D*. — The bearing surface between the upper chord members is inclined 63½ degrees with the fibers, making the

allowable unit-compression 1500 pounds per square inch. Since the horizontal component of the maximum chord stress is 27 200 pounds, the vertical bearing area needed is 27 200/1500 = 18.13 square inches. Since the bolt hole reduces the horizontal width of the bearing by $1\frac{3}{4}$ inches, the height required is $18.13/(6-1\frac{3}{4})$

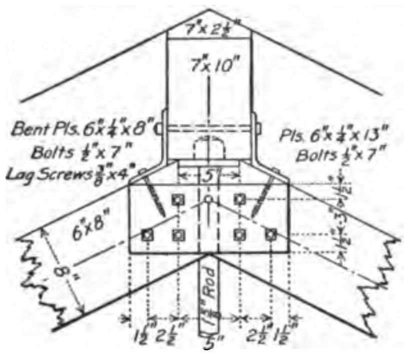


FIG. 62*f*. Joint *D*.

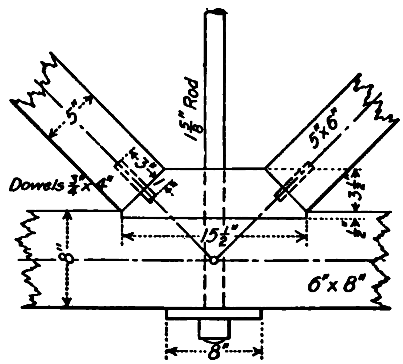


FIG. 62*g*. Joint *d*.

= 4.27 inches if the 8-inch side of the chords is placed vertical, or 2.90 inches if the 6-inch side is so placed. Much more than these values are provided.

JOINT *d*.—The bearing surface on each side of the angle block is inclined 45 degrees to the fibers, making the allowable unit-compression 1060 pounds per square inch. The area required is 15 300/1060 = 14.43 square inches, while that furnished is 5 × 6 = 30 square inches. The indent must provide sufficient bearing against the angle block to resist the horizontal component of the greatest difference between the stresses in the diagonal struts which bear against the block. This difference is 2860 pounds, or the horizontal component of the greatest wind stress in one strut. The depth of indent must then be at least 2860/(1800 × 6) = 0.265 inch; but it is better to make it $\frac{1}{2}$ inch. The bearing on the base of the block must resist the full stress in the vertical rod or 20 530 pounds. The required bearing area is therefore 20 530/330 = 62.21 square inches. As the block is

over 12 inches long and 6 inches wide, there is some excess of area furnished. Dowels are inserted to hold the struts in position, as shown in Fig. 62*g*.

JOINT *c*.— In a similar manner to that explained for joint *B*, the approximate depth of indent is found to be 1.21 inches, and after revision it is made 1¼ inches deep, since 1⅛ inches is not quite large enough.

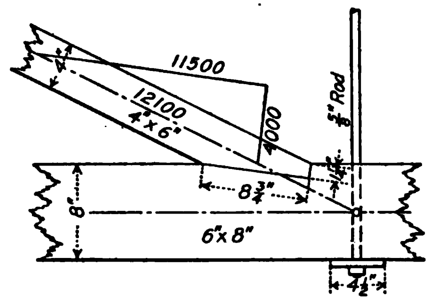


FIG. 62*h*. Joint *c*.

If the 8-inch side of the upper chord is placed horizontally, the ends of the struts will be housed into the chords, and the timber outside of the housing will resist any lateral displacement of the struts. If, however, the 6-inch side is placed horizontally, each end of the strut will be held in position by a ½-inch lag screw 6 inches long. At *d* a 1-inch dowel, 5 inches long, is inserted to hold the strut in position.

ART. 63. DESIGN OF END JOINT.

The order of design given in Art. 58 applies more particularly to the type shown on Plates I and II. An alternative design of that type for the truss under consideration in this chapter is presented later in this article. Since steel-plate connections are efficient and make simple details, the first design is to be one containing steel side plates with flats riveted to them, which act in exactly the same manner as in the tabled fish-plate joint. On each chord the tables which receive the compression are placed perpendicular to the direction of the stress, so that the bearing is squarely on the ends of the fibers. This design avoids all eccentricity in the joint or secondary flexure in either chord member.

The maximum stress in the upper chord is 50 680 pounds. The bearing area required is 50 680/800 = 23.16 square inches, and the required shearing area is 50 680/240 = 211.17 square inches. It is more economical to place the chord with its 8-inch side vertical, since it increases the length of the bearing surfaces and shortens the shearing surfaces. Assuming two tables on each side of the chord, the thickness of the flats composing the tables is $28.16/(4 \times 8) = 0.895$ inch. This is but slightly more than $\frac{7}{8}$ inch, and since there is considerable resistance due to the friction of the upper on the lower chord timber, the thickness of $\frac{7}{8}$ inch will be adopted. The clear distance between the tables equals the shearing length for one table, making it $211.17/(4 \times 8) = 6.599$ or $6\frac{3}{4}$ inches, which allows for the bolt holes to be deducted from the shearing surface.

The pressure against one table is 12 670 pounds, which, according to a handbook, requires three $\frac{3}{4}$-inch rivets in single shear to resist it. In order to provide sufficient bearing area against the rivets the large connecting plates must be $\frac{5}{16}$ inch thick. The width of the flats should be at least $2\frac{1}{2}$ inches (Art. 21), but to reduce the lateral pressure due to the moment of rotation on the tables let the width be increased to 3 inches. The outer rivets will be placed $1\frac{1}{2}$ inches from the end of the flat, making the spacing between the rivets $2\frac{1}{2}$ inches.

Each plate has to be investigated as a column between the sections where the bolts hold it in position. The radius of gyration of the plate is $r = 0.289 \times 5/16 = 0.0903$ inch, and since the length between centers of tables is 9.75 inches, $l/r = 9.75/0.0903 = 108$, which is less than the allowable limit of 120. The thickness of the plate, therefore, need not be increased.

The moment of rotation on each table is $12\,670(0.3125 + 0.875) = 7520$ pound-inches. The stress in each of the two bolts next to the table which are $3\frac{1}{2}$ inches from the bearing

surface is $7520(2 \times 3.5) = 1075$ pounds. This requires a bolt only $\frac{7}{16}$ inch in diameter, but since the bolts have additional duty in holding the plates in position, the diameter is increased to $\frac{1}{2}$ inch. To hold the upper edges of the plate in contact with the timber two track spikes $\frac{5}{16}$ by $2\frac{1}{2}$ inches may be employed.

In a similar manner the connections with the lower chord timber are designed. Assuming a depth of 8 inches for the timber, the flats may be $\frac{13}{16}$ inch in thickness, but they will be made $\frac{7}{8}$ inch, the same as for the upper chord. The length of the shearing surface required is 6.195 or $6\frac{1}{4}$ inches. Let this

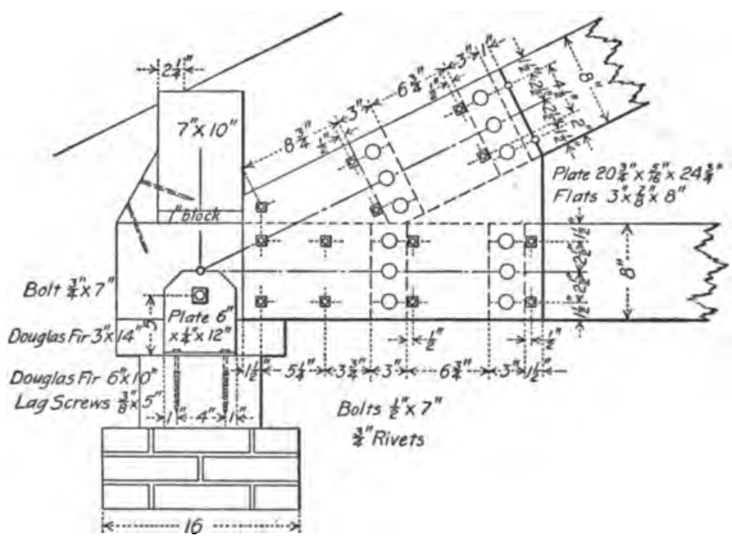


FIG. 63a. End Joint a.

distance also be made the same as for the upper chord, or $6\frac{3}{4}$ inches. In this case the bolts must be placed on the opposite side of the tables, and hence the end table is placed farther from the edge of the plate than at the upper chord timber. For bolt holes $\frac{9}{16}$ inch in diameter, the net section of the 6 by 8 timber is $(6 - 2 \times 0.875)(8 - 2 \times 0.5625) = 29.22$ square inches, which is

slightly in excess of that required (Art. 60) Other dimensions and details are given in Fig. 63*a*.

It is also found by computation that the lower chord has sufficient excess of strength at the middle of the end panel to carry the future ceiling load as a uniform load between panel points. As the same section is continued throughout and the stresses are less in the other panels, the entire ceiling load may be supported in this manner, if desired.

For a joint of the type indicated on Plates I and II, it is more economical to place the longer side of the upper chord section horizontally. This joint is designed on the principle that, owing to the shrinkage of timber in drying out or seasoning, the bolts shown in Fig. 63*b* cannot take any part of the horizontal component of the stress in the upper chord. Their function is

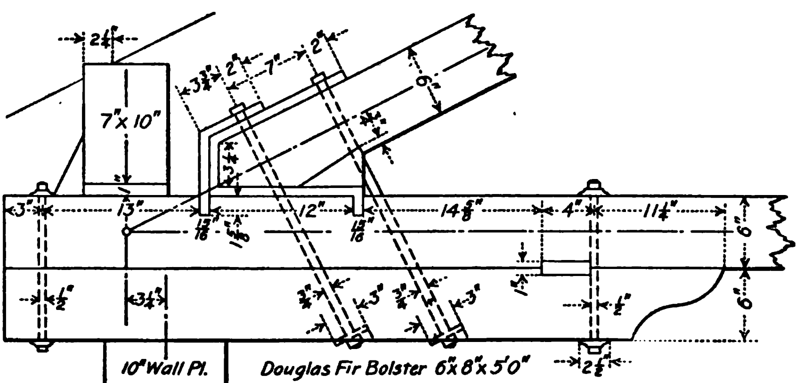


FIG. 63*b*. End Joint *a*.

merely to hold the parts firmly together. A diameter of $\frac{3}{4}$ inch may therefore be assumed as giving ample strength. The net tensile strength of each bolt is $0.302 \times 15\,000 = 4530$ pounds, requiring a net bearing area for the washer on the bolster of $4530/(1.25 \times 620) = 5.85$ square inches, since the allowable stress on the section making an angle of $26\frac{1}{2}$ degrees with the fiber is 620 pounds per square inch, by Fig. 62*b*. The gross

area is $5.85 + 0.60 = 6.45$, which requires a plate washer 2.54 inches square. The standard size is 3 inches. The bolts are expected to be drawn up to their full safe tensile strength.

The entire horizontal component of the stress in the upper chord must be resisted by the lugs which engage notches in the lower chord, and also by the shoe plate at the toe. Since the pressure of the plate on the side of the fibers increases their resistance, the unit compressive stress at the toe may be taken the same as on the ends of the fibers. The depth of toe needed is therefore $45\,300/(1800 \times 8) = 3.15$ or $3\frac{1}{4}$ inches. The horizontal bearing of the upper chord timber required is $22\,700/620 = 36.6$ square inches, and its length $36.6/8 = 4.58$ or $4\frac{5}{8}$ inches, which is exceeded by the length furnished.

If two lugs are employed, their depth of bearing must be $45\,300/(2 \times 1800 \times 8) = 1.573$ or $1\frac{5}{8}$ inches. Assuming the thickness to be $\frac{15}{16}$ inch, the bending moment for the lug, treated as a cantilever under uniform load, is $22\,650 \times \frac{1}{2}(1.625 \times 0.9375) = 29\,020$ pound-inches. The resisting moment of the lug is $25\,000 \times 8 \times d^2/6$, and when equated with the bending moment gives $d = 0.933$ or $\frac{15}{16}$ inch as assumed.

The net shearing surface required between the lugs is $22\,650/240 = 94.5$ square inches. Allowing 1.5 square inches for the bolt holes, the distance between lugs is $(94.5 + 1.5)/8 = 12$ inches. The same distance is needed from the outer lug to the end of the chord. The net width of the lower chord is $8 - 0.875 = 7.125$ inches. Since the horizontal component of the wind reaction can be transmitted to the chord through the bolster and its key, the net tensile area at the inner lug needs only to be $45\,300/1650 = 27.5$ square inches. The net depth of the chord is then $27.5/7.125 = 3.86$ inches, making the gross depth $3.86 + 1.625 = 5.49$ or 6 inches.

A bolster is added at the end joint, both to avoid cutting the lower chord on account of the washers for the inclined bolts,

and to stiffen the chord timber, which is subject to secondary bending on account of eccentricity in the applied forces. The key connecting the bolster should be designed to transmit the horizontal component of the full safe tensile stress in the inclined bolts as well as the horizontal component of the reaction at the support due to wind pressure on the truss. This gives a force equal to $2 \times 4530 \times \sin 26^\circ\ 34' + 2300 = 6360$ pounds, hence the depth of the key is $2 \times 6360/(1800 \times 8) = 0.884$ or 1 inch. The length needed to resist the shear parallel to the fibers, and that to resist the moment of rotation, are found by the method explained in Art. 16 to be $3\frac{5}{8}$ and $2\frac{3}{4}$ inches respectively. Let the length be made 4 inches. The size of one bolt required to resist the vertical reactions of the key is $\frac{7}{16}$ inch, but since the timber is over 6 inches wide and for the reason indicated at the beginning of this paragraph two $\frac{1}{2}$-inch bolts are adopted. The diameter of the ogee washer is $2\frac{1}{2}$ inches.

Since the notch at the inner lug cuts the fibers to a depth of $1\frac{5}{8}$ inches, the stress transmitted by those fibers must be transferred down to the lower fibers on the left of the key and which continue past the net section at the lug. The upper fibers can carry a stress of $6.875 \times 1.625 \times 1650 = 18\,430$ pounds. The fibers above the key which transmit its stress of 6360 are $6360/(1650 \times 6.875) = 0.56$ inch deep, and these require a shearing length on the left of the key of $0.56 \times 1650/240 = 3.85$ inches. The total depth of fibers on the left of the key to which stresses can be transmitted is 1.56, the working strength of which is $1.56 \times 7.125 \times 1650 = 18\,340$ pounds. The total length of shearing surface between the inner lug and key must be $18\,340/(240 \times 7.125) = 1.073$ inches. Therefore, the required clear distance between the inner lug and key is $3.85 + 10.73 = 14\,58$ or $14\frac{5}{8}$ inches. It should be noted that since the upper fibers extend to the right, the net width is determined by the two $\frac{9}{16}$-inch bolt holes; while the lower fibers

extend to the left, in which case the net width is determined by one $\frac{7}{8}$-inch bolt hole.

The preceding design may be modified so as to reduce the cost materially, by substituting for the inner lug a flat riveted to the plate as illustrated in Fig. 63*c*. In this the thickness of the plate is not determined by flexure and may therefore be reduced in thickness more than 50 percent. Since only three rivets can be inserted in a single row across the chord, each rivet must resist a stress in single shear of $22\,650/3 = 7550$ pounds. This requires a 1-inch rivet. The plate must have sufficient thickness for the same stress in bearing on the rivet, which must, therefore, be $\frac{7}{16}$ inch, but this is increased to $\frac{1}{2}$ inch, as the rivet heads have to be countersunk. The width of the flat should be made $3\frac{1}{2}$ inches (Art. 21).

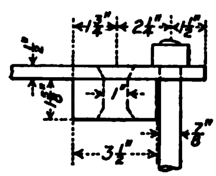


FIG. 63*c*.

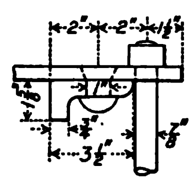


FIG. 63*d*.

A further improvement may be made by substituting an angle for the flat, cutting down its shorter leg to the width of $1\frac{5}{8}$ inches (Fig. 63*d*). The smallest angle having the necessary thickness is $3\frac{1}{2}$ by $2\frac{1}{2}$ inches (see Cambria). Let its thickness be assumed as $\frac{3}{4}$ inch. The bending moment in the horizontal leg of the angle is $22\,650 \times \frac{1}{2}\,(1.625 - 0.75) = 9910$ pound-inches. The resisting moment is $15\,000 \times 8d^2/6$, which, on being equated with the bending moment, gives $d = 0.704$ or $\frac{3}{4}$ inch as assumed. The moment couple which causes rotation is $22\,650 \times \frac{1}{2}(1.625 \times 0.4375) = 23\,360$ pound-inches, and since the bolt is 4 inches from the vertical bearing surface of the angle, the stress in the bolt is $23\,360/4 = 5840$ pounds, which requires a diameter of $\frac{7}{8}$ inch.

Prob. 63. Design a forged steel shoe and its connecting bolts which shall provide three bearing surfaces for the timber instead of two.

ART. 64. SUPPORTS AND SPLICES.

The purlins may be held in position by various means. At the peak two bent plates $\frac{1}{4}$ inch thick are each fastened to the chord by two lag screws $\frac{5}{8}$ by 4 inches, and a $\frac{1}{2}$-inch bolt holds the purlin in place. For the intermediate purlins the simplest connection consists of a wooden triangular block and spikes. The component along the chord of the maximum pressure from the purlin is 2190 pounds. This requires three 60d wire nails or spikes, since the table in Art. 6 gives the strength at the yield point for one nail as $0.60 \times 2000 = 1200$ pounds. The resistance of one nail is then about 60 percent of the strength at the yield point, which is safe. Two nails may also be driven through the block into the purlin, and one or two through the side of the purlin into the chord timber. The purlin need not then be notched over the chord. The weakening effect of notching is discussed in Art. 37.

Upon investigation the intermediate purlins are found to have more than the necessary bearing area on the chord. The one at the peak requires a bearing area of $(4890 \times 1440)/330 = 19.2$ square inches. The area of a round hole with a diameter equal to the long diameter of the $3\frac{1}{4}$-inch nut is 16.8 square inches. The gross area is then $19.2 \times 16.8 = 36$ square inches. As the purlin at the peak is only 5 inches wide, the length of bearing is about 8 inches. It is necessary to nail blocks aside of the plate washer so as to give a bearing for the full width of the upper chord.

The vertical component of the maximum reaction is 26 570 pounds, which requires a bearing area of $26\,570/330 = 80.4$ square inches. For a lower chord width of 6 inches, the width of the wall plate must be $80.4/6 = 13.4$ or 14 inches. As this is relatively a very large width, it may be reduced by inserting a block of Douglas fir and using a Douglas fir wall plate, in which case the required width of the wall plate is $26\,570/(465 \times 6) =$

9.52 or 10 inches. In the alternative design the bolster is made of Douglas fir.

If the truss is supported by a wall of common brickwork, the allowable pressure per square inch is about 100 pounds. A bearing surface is required of $26\,570/100 = 265.7$ square inches, and for a width of 10 inches the length is 26.57 or 27 inches. Let a length of 28 inches be taken. The thickness may be approximately computed by treating each end of the 28-inch wall plate as a cantilever under uniform load. The result is $4\frac{3}{8}$ or 5 inches. Let a depth of 6 inches be taken to secure increased stiffness. No definite computation can be made regarding the uplifting effect of the wind. Two anchor bolts $\frac{3}{4}$ inch in diameter and 24 inches long will have ample strength for the purpose.

The truss is held in position on the wall plate by a connection of bent plates, lag screws, and bolt, as shown in Fig. 63*a*. The horizontal force to be resisted is the horizontal component of the reaction due to wind pressure. In the alternative design the bolster may be notched down over the wall plate to resist this pressure.

In the first design of the end joint there is no secondary bending in the lower chord such as occurs in the alternative design. An approximate determination of the bending moment near the end of the lower chord due to its eccentric tension may be made in the following manner: The pressure on the upper chord may be resolved into its two components; the vertical component is a resultant of the distributed pressure on the lower chord and is applied at the middle of the bearing surface. The horizontal component acts upon the lower chord as a resultant of the pressure on the lugs, while the resultant of the tension in the lower chord is applied at the middle of the uncut depth below the lugs. The moment of rotation is therefore $45\,300 \times 3 = 135\,900$ pound-inches, and in order to counterbalance this moment the reaction of the support should be applied at a dis-

tance to the left of the center of bearing of the upper chord timber equal to 135 900/22 700 = 6.0 inches. But the center of support is 9.3 inches distant, therefore the bending moment to be resisted by the bolster is 22 700 × 3.3 = 74 900 pound-inches. Equating this to the resisting moment of a Douglas fir bolster of depth d, the depth is found to be 5.587 or 6 inches. When the bolster is firmly bolted to the chord it relieves the chord timber of the larger part of its flexure. If the upper surface of the bolster is slightly curved, thus reducing the depth at the ends, it may relieve the flexure in the chord timber entirely when the end bolts are tightened.

The splice in the upper chord should be made near the quarter point of the panel length, which is assumed to be the location of the point of inflection in the chord timber acting as a column. The half-lap joint is an effective one for the purpose, and is given the proportions indicated in Art. 27.

If the truss is to be hoisted into position by a derrick, as is sometimes the case, the splice at the peak has to resist considerable tension, and should be designed in all its details to resist the computed erection stresses. An effective joint may be secured by metal plates and bolts. The metal plates may be replaced by wooden planks if desired.

The splice in the lower chord is preferably placed at the center of the span, since the length of each stick will not exceed a car length. If, however, the available timber is not so long, the joint should be placed in the panel cd (Fig. 60a) as close to the panel point c as convenient, due regard being paid to commercial lengths of lumber. As steel connecting plates were adopted for the end joint in the first design, a tabled fish-plate joint with steel plates may be appropriately selected. For the alternative design, wooden fish plates may be adopted. Since complete designs for fish-plate joints are given in Chapter II, no details regarding the lower chord splice will be given in this article.

ART. 65. ANALYSIS OF WEIGHT.

The following bills of material include one roof truss and a single bay of the roof, or that portion which is supported by one intermediate truss. All the wood is Western hemlock except a few pieces marked as Douglas fir, the weight of both species being taken as 3 pounds per foot board measure.

MATERIAL FOR ONE TRUSS.

DESIGNATION.	NO. OF PIECES.	SECTION.	LENGTH.	FEET B.M.	WEIGHT IN LBS.
Timber:					
Upper chord	2	6″ × 8″	24′ 0″	192	576
Upper chord	2	6″ × 8″	10′ 5¹″ [11′]	88	264
Lower chord	2	6″ × 8″	30′ 7″ [32′]	256	768
Strut *Bc*	2	4″ × 6″	10′ 6½″ [11′]	44	132
Strut *Cd*	2	5″ × 6″	10′ 4¼″ [11′]	55	165
Block at *a*	2	1″ × 7″	6″	1	3
Block at *a*	1	3½″ × 6″	6″	1	3
Blocks at *B* and *C*	2	6″ × 6″	6″	3	9
Block at *D*	1	2½″ × 7″	6″	1	3
Block at *d*	1	4″ × 6″	1′ 3½″	3	9
Douglas fir block at *a*	2	3″ × 6″	1′ 2″	4	12
Douglas fir wall plate	2	6″ × 10″	2′ 4″	23	69
				671	2013
				WEIGHT PER FT.	
Steel Rods:					
Rod *Bb*	2	⅝″ O	5′ 9″	1 043	12
Rod *Cc*	2	1″ O	10′ 10″	2.670	58
Rod *Dd*	1	1⅝″ O	15′ 11½″	7.051	112
Nuts:					
4 for ⅝″ rod; 4 for 1″ rod; 2 for 1⅝″ rod,					13
Plate Washers:					
2 at *B*, 2″ × 2″ × 5/16″; 2 at *C*, 3½″ × 3½″ × ⅜″; 1 at *D*, 5″ × 6″ × ⅝″; 1 at *d*, 7¼″ × 7¼″ × 9/16″; total weight,					17
Cast-iron Ogee Washers:					
2 at *c*, 5″ diam.; 2 at *b*, 3″ diam.; 8 for half-lap joint, 3″ diam.,					14

Steel Plates:

				POUNDS.
At *a*,	4 plates,	20¾″ × 5/16″ × 24¾″	@ 22.05 lb.,	182
	16 flats,	3″ × ⅞″ × 8″	@ 8.93 lb.,	96
	4 bent plates,	6″ × ¼″ × 12″	@ 5.10 lb.,	20
At *D*,	2 plates,	6″ × ¼″ × 13″	@ 5.10 lb.,	11
	2 bent plates,	6″ × ¼″ × 8″	@ 5.10 lb.,	7

				Pounds.	Weight in Lbs.
Fro splice in lower chord:					
At d,	2 plates,	8″ × ¼″ × 33¼″	@ 6.80 lb.,	38	
	8 flats,	2¼″ × 1⅛″ × 8″	@ 5.26 lb.,	33	387

Steel Bolts including Nuts:

At a, 26 bolts, ½″ × 7″, 2 bolts, ¾″ × 7″; at D, 1 bolt ½″ × 8″, 6 bolts ½″ × 7″; for half-lap joint, 4 bolts, ¾″ × 10″; for splice in lower chord, 10 bolts, ¾″ × 7″; total weight, 28

Lag Screws:

At a, 8 – ⅜″ × 5″; at B, 2 – ⅜″ × 5″; at C, 2 – ⅜″ × 5″; at D, 2 – ⅜″ × 4″; at c, 2 – ⅜″ × 4″; total weight, 3

Total weight of one truss, 2657

WEIGHT OF ONE BAY OF ROOF, 12 FEET LONG.

	Pounds.
7 Purlins, 7″ × 10″ × 12′, 490 ft. B.M., weight,	1 470
12 Rafters, 2″ × 6″ × 23′ 10¾″ [24′], 288 ft. B.M., weight,	864
12 Rafters, 2″ × 6″ × 14′ 6¾″ [16′], 192 ft. B.M., weight,	576
Sheathing, 1″ thick, 12′ × 76′, 912 ft. B.M., weight,	2 736
Sheeting, etc., for cornice, 210 ft. B.M., weight,	630
Slate, 12″ × 20″ × $\frac{3}{16}$″, 8½″ exposed when laid, 912 sq. ft. @ 6½ lb.,	5 928
Roofing nails, 4d galvanized,	19
Nails for rafters, 10d, 2 lb.; for purlins, 60d, 2 lb.,	4
Nails for sheathing and cornice, 8d, 21 lb., 20d, 2 lb.,	23
Total,	12 250

The weight of the overhanging portion of the roof including the cornice is 1450 pounds, hence the net weight for 6 panels of the roof truss is 10 800 pounds. On adding the weight of the truss the average dead panel load is $(10\,800 + 2657)/6 = 2243$ pounds. The panel dead load assumed in finding the stresses in the roof truss exclusive of the ceiling is 2 490 pounds (Art. 60), which is on the safe side.

The weight of the roof truss according to the formula used is 3600 pounds, while the actual weight is only about 2660 pounds. This difference is larger than it should be, and indicates that for the material, loading and unit-stresses adopted for this truss, the second constant in the parenthesis of the formula is too large. The formula given in Roofs and Bridges, Parts I and II, is

$W = \frac{1}{2} al(1 + 0.1\,l)$, making the weight of the truss under consideration 2520 pounds, which is somewhat too small. It will probably give the weight closely for a design in which the heavy ceiling load is omitted.

The theoretic weight (Art. 55) of this truss is 2003 pounds, consisting of 1833 pounds of lumber and 170 pounds of steel rods. To obtain the actual weight of the truss members and of all details it is therefore necessary to add 32.6 percent to the theoretic weight. The weight of steel and cast-iron is 644 pounds, which is 32.2 percent of the actual total weight.

ART. 66. ESTIMATE OF COST.

The approximate cost of the roof truss according to the first design, in which vertical steel plates with flats riveted to them are employed at the end joints, and similar fish plates are used for the splice in the lower chord, is estimated as follows:

Lumber, 671 ft. B.M. @ 2¢	$13.42
Steel rods and nuts, 195 lb. @ 3¢	5.85
Steel plates and washers, 195 lb. @ 4¢	16.16
Steel bolts and nuts, 28 lb. @ 3¢	.84
Lag screws, 12 @ 4¢; 4 @ 3¢	.60
Cast-iron ogee washers, 14 lb. @ 2¢	.28
Framing and erection, 671 ft. B.M. @ 2½¢	16.78
Total	$53.93

The estimated cost for the alternative design in which forged steel shoe plates are used at the end joints, the 8-inch side of the cross-section of each chord being therefore placed horizontally, and with timber fish plates in the lower chord splices, is as follows:

Lumber, 731 ft. B.M. @ 2¢	$14.62
Steel rods and nuts, 197 lb. @ 3¢	5.91
Steel plates and washers, 241 lb. @ 4¢	9.64
Steel bolts and nuts, 44 lb. @ 3¢	1.32
Lag screws, 12 @ 4¢; 4 @ 3¢	.60
Cast-iron washers, 19 lb. @ 2¢	.38
Framing and erection, 731 ft. B.M. @ 2½¢	18.28
Total	$50.75

In the following estimate of cost for material in one bay of the roof, one-tenth is added to the superficial area to allow for waste in the sheathing, and one-fifth is added to the lumber in the cornice. The cost of carpenter work includes that of nails.

Lumber, 2225 ft. B.M. @ 2¢	$44.50
Laying the sheathing, 91 squares @ 60¢	5.46
Framing and placing purlins and rafters, 970 ft. B.M. @ 1¢	9.70
Framing, etc., of cornice, 252 ft. B.M. @ 1¢	2.52
Slates, 9.5 squares @ $8.00	76.00
Labor for slating, 47.5 hr. @ 35¢	16.63
Roofing nails, 19 lb. @ 3¢	.57
Total	$155.38

The total cost therefore of one truss and the roof which it supports is $209.31 for the first design, and $206.13 for the second design.

The preparation of bills of material in a proper form, and of detailed estimates of cost, is a subject to which the young designer should give especial study. There is comparatively little material of this kind published in the form in which it is used in practice. As an interesting example of a comparative estimate for three types of construction, refer to an article on Round-house Framing by R. D. COOMBS, in Transactions American Society of Civil Engineers, vol. 55, page 157, December, 1905; or Railroad Gazette, vol. 38, page 624, June 9, 1905. For an example of the analysis of cost for a structure, but in which lumber was used only for falsework and forms, see an article on Arch Rib Bridge of Reinforced Concrete at Grand Rapids, by GEORGE JACOB DAVIS, Eng. News, vol. 55, page 321, March 22, 1906.

ART. 67. DETAIL DRAWINGS.

As many of the details for the design of the roof truss made in this chapter have been illustrated in preceding articles, the working drawing will be omitted. In its stead are given Plates I and II, which are reproduced by lithography from the original

detail drawings prepared by two students as a part of their regular class work.

The design of C. L. TODD is for a span of 82 feet, with a rise equal to one-fourth of the span, the trusses being spaced 14 feet. It will be observed that the steel shoe plate has three bearing surfaces in the lower chord, the middle one being provided by an angle riveted to the plate. The general arrangement of details, the dimensions required for construction, etc., are all clearly shown and require no additional description.

The design of R. M. BOWMAN is for a span of 80 feet, with a rise of one-fourth of the span, the trusses being spaced 12 feet apart. The truss is of the Belgian type, like Fig. 54*b*, in which the struts are perpendicular to the upper chord. The beveled washers shown are all of the same type. The flat riveted to the steel shoe plate is connected by two rows of staggered rivets. In both cases the designs were made for longleaf yellow pine, the unit-stresses adopted differing somewhat from those for the same wood used in this text-book.

Plates III and IV give a part of the general and detail drawings of a roof truss for a first-class freight depot. The plans were originally prepared as a standard, but later they were superseded by special plans prepared for each location. One of the distinctive features consists in the adoption of cast-iron angle blocks with projecting lugs and pins, so that the braces are all cut square at each end, with a hole bored to receive the projecting cast pin. The end joint has double steps. The lower chord is built up of five 2 by 10-inch planks, arranged to break joints as indicated on Plate IV. It will be observed that all the joints are concentrated in the two panels adjacent to the center, without exceeding a length of 24 feet for the planks. The splice bolts act as beams in transmitting the stresses from the end of one plank to the adjacent planks which continue past the joint. The bolster is cogged (Art. 28) over the wall plate and is con-

tinued beyond the connection of the angle brace, which is a housed mortise-and-tenon joint. The braces of the overhang on the outside have mortise-and tenon joints without the housing.

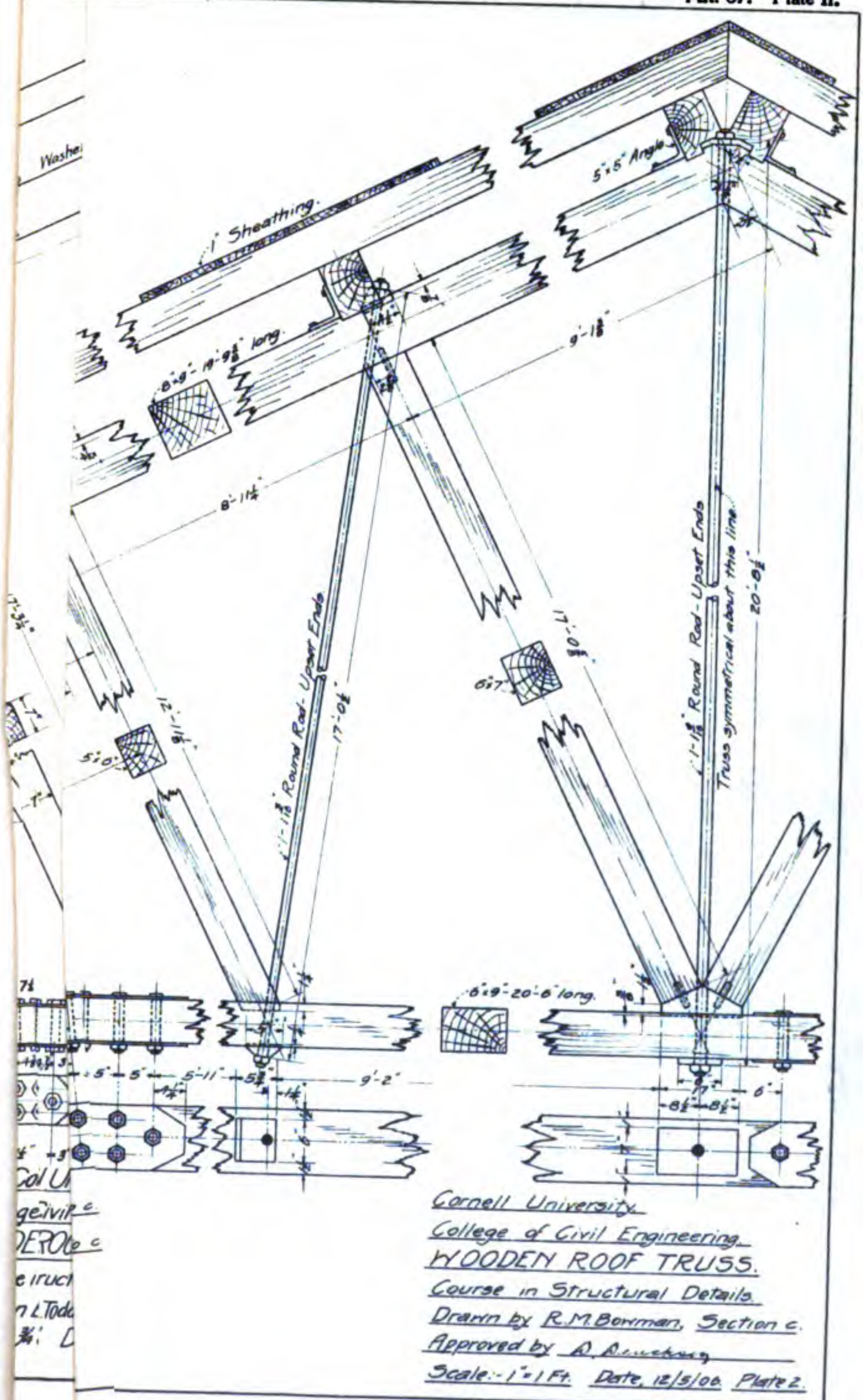
The roof is arranged to dispense with common rafters. The purlins are small, in section like joists or rafters, and spaced close together, the sheathing being laid directly on the purlins. At the peak two purlins of increased depth are spiked together, and take bearing on top of the bent-plate washer.

Prob. 67. Estimate the difference in cost for the alternative composition of the lower chord as given in the note on Plate III.

ART. 68. TESTS OF END JOINTS.

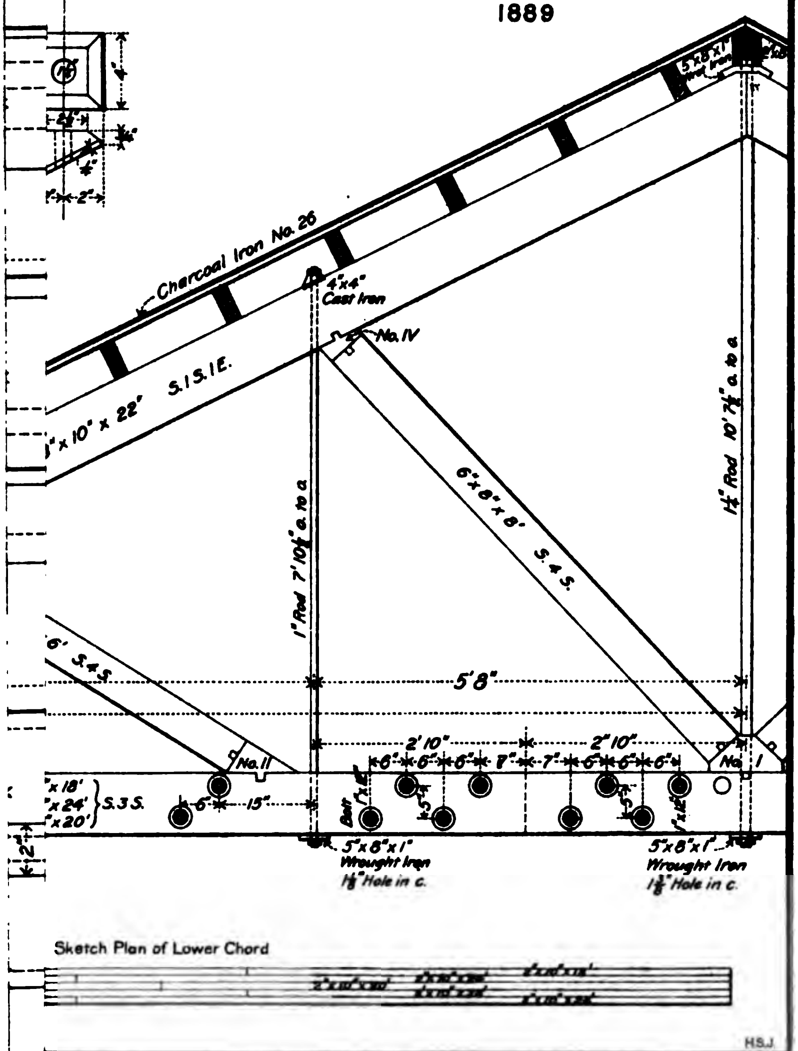
Ten full-size tests to destruction of different kinds of end joints in roof trusses were made in the engineering laboratory of the Massachusetts Institute of Technology, the detailed results of which, accompanied by dimensioned drawings and half-tone illustrations of the manner of failure, were published in Technology Quarterly, September, 1897, and subsequently reprinted in pamphlet form. Figs. 68*a–l* are reprinted from a review of these tests by F. E. KIDDER in Engineering Record, vol. 42, page 464, Nov. 17, 1900. Apart from the half-tone illustrations, this article is the more valuable for reference, since the author gives the description copied from the records, and the results of his own computations of unit-stresses in the connections, deductions, and conclusions. See also page 624 in the same volume for additional discussion on the subject.

Fig. 68*a* indicates the form in which the trusses were tested, the arrows indicating the points where the load was applied and the truss supported. The timber was hard pine in all cases. The most important result obtained from this set of experiments is that the maximum resistance of the timber to horizontal shear beyond the step joint and that due to compression on the side of the upper chord timber by through bolts or straps do not

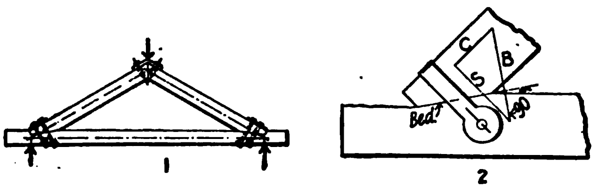

1" Sheathing.
5"×5" Angle
8"×9" - 19'-9⅝" long.
9'-1⅛"
8'-11¼"
1"-1⅛" Round Rod - Upset Ends
17'-0¼"
6"×7"
17'-0⅜"
1"-1⅛" Round Rod - Upset Ends
Truss symmetrical about this line.
20'-0½"
5"×6"
6"×9" - 20'-6" long.
9'-2"
Cornell University.
College of Civil Engineering.
WOODEN ROOF TRUSS.
Course in Structural Details.
Drawn by R.M.Bowman, Section c.
Approved by
Scale:- 1"=1 Ft. Date, 12/5/06. Plate 2.

Northern Pacific Railroad

Detail Plan
of a
ROOF TRUSS
for a
First Class Freight Depot
1889



occur simultaneously. In other words, the joint and fastenings, or two sets of fastenings, do not work in harmony. The lesson to be learned from the tests by designers is that the joint

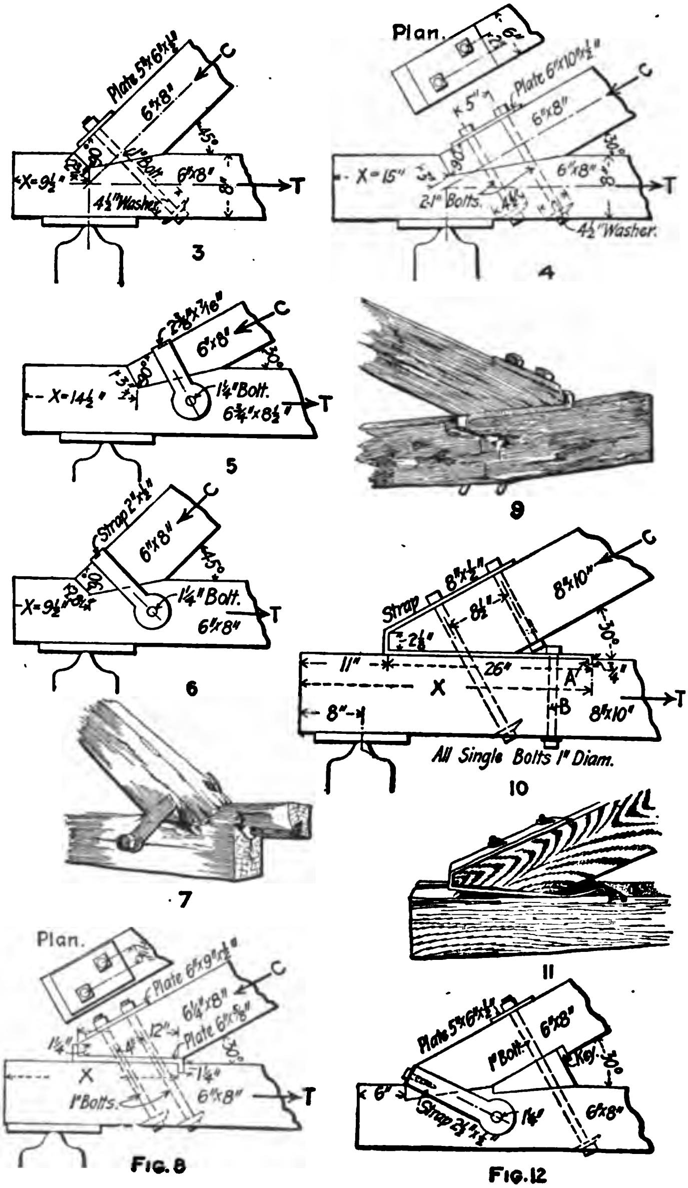


FIGS. 68*a*, *b*. Tests of End Joints of Roof Trusses.

should be designed so as to depend either on one or the other element of resistance.

The disadvantage of the resistance of inclined bolts or straps to resist the horizontal component of a given stress in the upper chord is indicated in Fig. 68*b*; the stress in the bolt or strap must be considerably larger than the component force to be resisted, and since the bearing is applied on the side of the fiber, weak elements of the timber are brought into action, for the timber is weak in its resistance to compression on the side of the fibers, and yields considerably at the same time.

The two forms of shoe are shown to be superior to the step joints. This is largely due to insufficient provision for lengths of shearing surfaces; but herein lies one of the advantages of the shoe, since the shearing surface can be extended inside of the toe, for usually the distance outside of the toe is limited by other conditions of the structure. The shoe in Fig. 68*h* (No. 8) would have had considerably increased strength if the toe had been deeper and the shoe plate had been continuous with the upper bearing plate. The shoe in Fig. 68*j* would also have been materially stronger if a hook bolt had been used directly over the lug. As the resistance of the lug depends upon its strength in flexure, the strength of the plates in the two cases are to each other as 25 is to 16.



FIGS. 68*c*–*l*. Tests of End Joints of Roof Trusses.

Nine tests of full-size wooden trusses with spans from 24 to 32 feet were made under the direction of G. K. SCOTT MONCRIEFF, the results of which are given by him in an article in Journal of Royal Institute of British Architects, vol. 6, page 113, Jan. 14, 1899. An abstract of this article was published in Engineering Record, vol. 39, page 284, Feb. 25, 1899. The object of the experiments was "to ascertain whether trusses, as constructed according to the dimensions generally accepted in English practice, had any advantage, from a practical point of view, over other designs more strictly in accordance with theory and more economical of material." One detail of design deserving especial mention which was emphasized by these experiments relates to the importance of using cylindrical bearing blocks of hard wood when strap or U-bolts are employed. In the end joint of trusses especially designed for the tests the bolts were placed in a horizontal position, instead of the usual inclination.

Four tests of triangular trusses with end joints like those in Fig. 63*b* were made in the laboratory of the College of Civil Engineering of Cornell University in 1902 by GUY E. LONG. The timbers were of longleaf yellow pine, having a compressive strength for short blocks of 7590 pounds per square inch, this value being the mean of 12 tests. The wrought-iron shoes gave no indication in any test of yielding. The plates were $\frac{3}{8}$ inch thick, and the timbers were 6 by 6 inches in section, the upper chords having an inclination of 30 degrees. The lugs engaged notches $1\frac{1}{2}$ inches deep. The effective span was 6 feet $1\frac{1}{4}$ inches, and the shearing length beyond each lug was $12\frac{1}{2}$ inches.

Since the span was so short, a difference in the deflection of the lower chord could be observed when blocks were used to hold down the inner lugs, and afterwards bolts were substituted for the same purpose. The most important facts obtained by this investigation relate to the effect of secondary flexure due to the eccentric application on the lower chord of the horizontal com-

ponent of the upper chord. A difference of $\frac{1}{2}$ inch each way in the location of the supports from the correct position could be detected by opposite deflections of the lower chord. The method of finding the position for the supports with respect to the center of bearing of the upper chord given in Art. 63 was confirmed by the tests.

In one case the truss was loaded with the supports at the intersection of the center lines of the chords until shearing occurred between the lugs. The supports were then placed directly beneath the outer lugs, and when the load was reapplied the lower chord deflected upward and split the timber inward from the bottom of the notch at the inner lug. The supports were then placed at the correct position, which is intermediate between the other two, and in this condition the truss supported a greater load than before, notwithstanding the two previous fractures of the timber at the joint.

In another test the truss was subjected to a load nearly 1.25 times that for which it was designed without showing any signs of weakness, the correct position for the supports having been found previously by trial on the basis of deflection. The supports were then moved outward $3\frac{1}{2}$ inches, and upon reloading the truss the timber sheared under a load of 76 percent of the previous one.

Prob. 68. Compute the working strength of the lower chord in Fig. 68*h* (No. 8); first, with respect to the shearing resistance of the timber; second, with respect to the compressive resistance of the toe; and third, with regard to the strength of the lug. Let the following working unit-stresses be assumed: Compression on the ends of the fibers, 1500; shear parallel to the fiber, 150; stress in the outer fiber of the lug, 20 000 pounds per square inch.

ART. 69. DETAILS OF ROOF TRUSSES.

Fig. 69*a* shows some characteristic details of a roof truss designed by HENRY GOLDMARK in 1902 for the freight-car shops of the Canadian Pacific Railway at Montreal. The truss has a

length out to out of 102 feet 2 inches supported by a column of steel at the middle. The depths over all at the ends and center are 7 feet 3 inches and 10 feet 9 inches respectively. The trusses are spaced 20 feet center to center. The illustration shows the splices of both upper and lower chords, as well

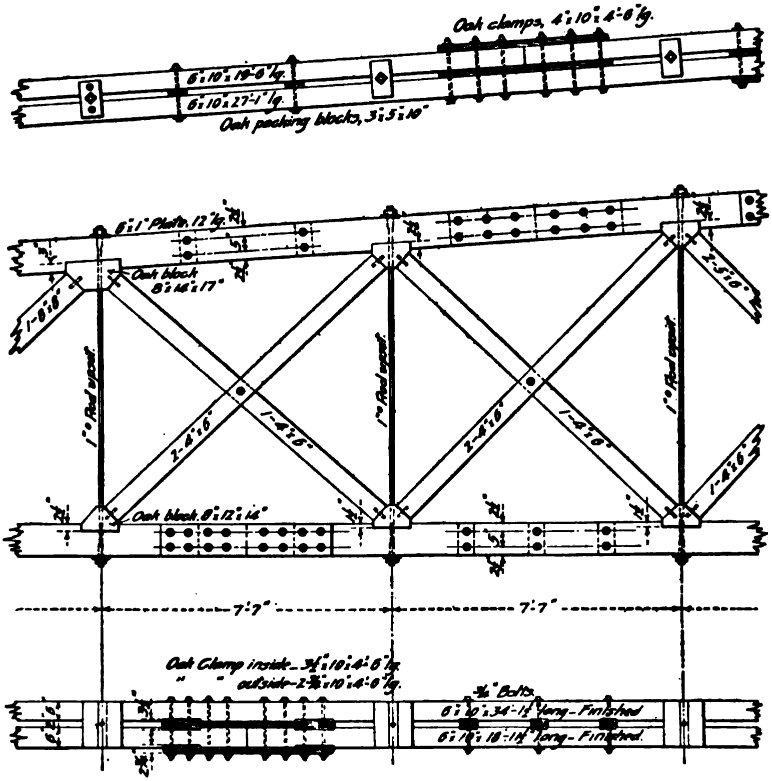


FIG. 69*a*. Part of Roof Truss for Freight-car Shops of Canadian Pacific Railway at Montreal.

as the spacing blocks between the timbers. Those in the lower chord are notched into both timbers to aid in equalizing the stresses between them. The braces are held in position by dowels against the oak angle blocks. For additional illustrations and their description see an article on Canadian Pacific

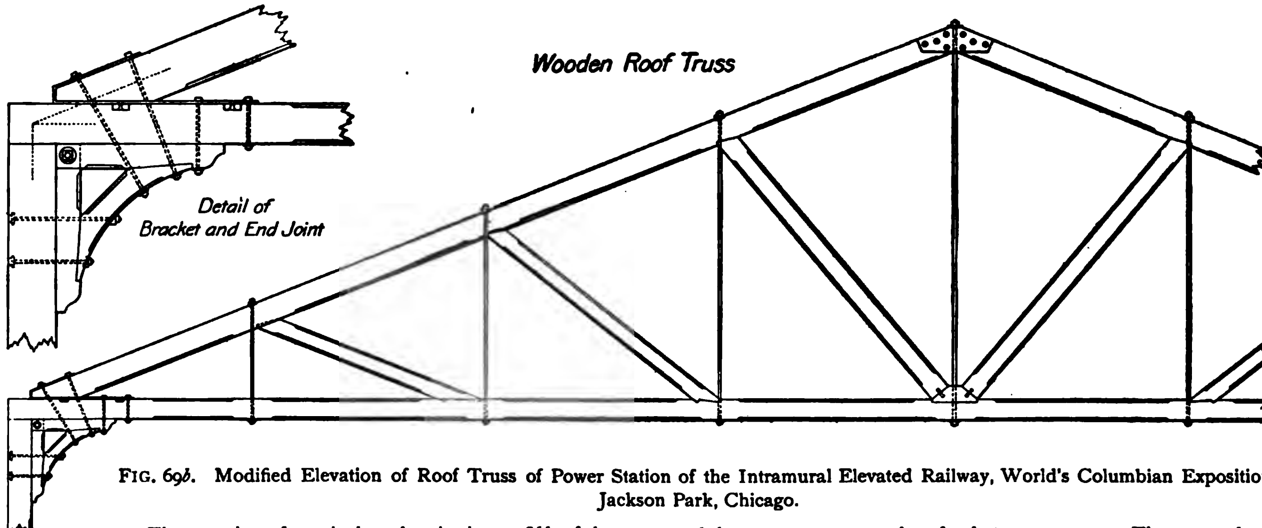


FIG. 69*b*. Modified Elevation of Roof Truss of Power Station of the Intramural Elevated Railway, World's Columbian Exposition, Jackson Park, Chicago.

The span is 77 feet 9 inches, the rise is one-fifth of the span, and the trusses are spaced 15 feet between centers. The upper chord is composed of 5 planks 2 by 12 inches, which break joints and are bolted together with $\frac{1}{2}$-inch bolts every 2 feet. The lower chord is built up in a similar manner of 2 by 10-inch planks. The braces are single sticks 8 by 8, 8 by 10, and 10 by 10 inches in section. The rods are $\frac{5}{8}$, $\frac{3}{4}$, $1\frac{1}{8}$, and $1\frac{3}{8}$ inches in diameter. The wrought-iron splice plates at the peak are fastened with $\frac{1}{2}$-inch bolts. The shoe plate is 10 by $\frac{5}{8}$ inches in section, and the inclined bolts are 1 inch in diameter. The truss is framed with a camber of $1\frac{1}{2}$ inches in the lower chord.

Railway Shops at Montreal; Part III. – Structural Features in the Blacksmith, Machine, Wood-working, and Other Shops, of Engineering Record, vol. 49, page 417, April 2, 1904.

At the end joint of the truss, the inclined brace has a single step engaging the lower chord, and a head block is placed outside, bearing against the inclined surface of the brace. A bolster is keyed to the bottom of the lower chord, and a long inclined bolt takes the thrust of the head block and reacts against the inner end of the bolster. The head block and bolster are also united by two vertical bolts passing between the lower chord timbers.

Figs. 69*c*, *d*, and *e* illustrate some details of roof trusses which were originally contributed to Engineering News in response to a request for criticism of a partial plan submitted by a correspondent. They are reprinted here for the purpose of indi-

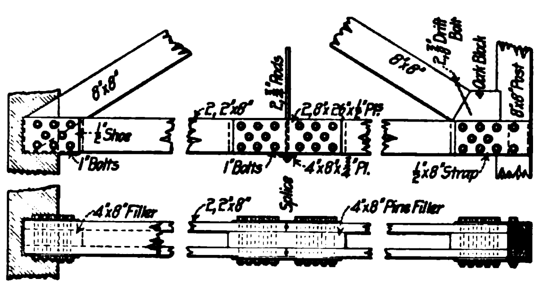


FIG. 69*c*.

cating certain defects in design which are prevalent in practice. The feature which first attracts attention in Fig. 69*c* is the plain fish-plate joint in the lower chord, in which three rows of bolts are placed in an 8-inch fish plate. Apparently no allowance has been made for the tendency of round bolts acting as beams to split the timber. The longitudinal distance between the staggered bolts of adjacent rows is so short that nothing seems to be gained by staggering them.

At the left end joint the shoe plate is bent twice at right

angles, and is supposed to resist bearing over the middle half of its horizontal end width. This condition involves flexure, for which a half-inch plate is inadequate. The joint would be improved by rounding the end bearing so as to reduce the flexure in the plate. Some of the bolts are too close to the edge of the upper chord timber and of the filler below it.

The design in Fig. 69*d* was offered as an improvement upon that in Figs. 69*c* and *e*. Its element of weakness consists in the apparent failure to consider the pressure on the side of the

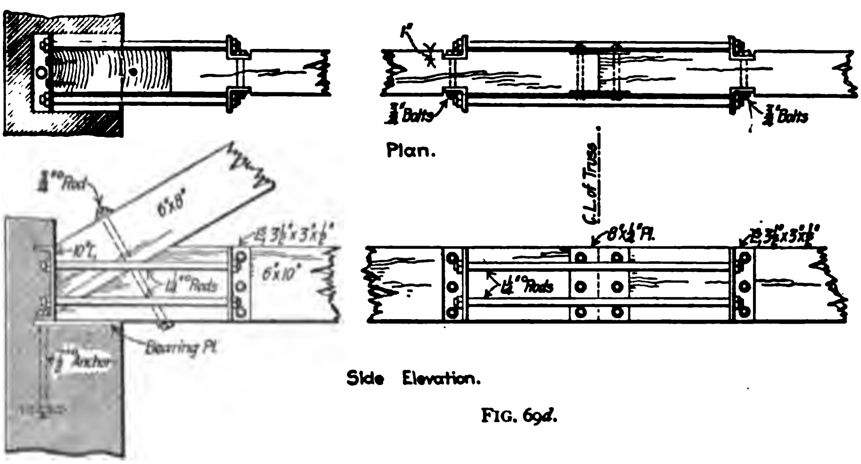


FIG. 69*d*.

fibers due to the moment of rotation in the angles. If each horizontal rod is properly designed, so far as the net section area due to direct tension is concerned, what probable stresses in the outer fiber of the net section may be caused by the uneven bearing of the nut on the angle?

In Fig. 69*e* attention is at once directed to the angle brace which is extended to the upper chord of the truss, and therefore introduces ambiguity into the stresses for a number of truss members. The truss becomes a statically indeterminate structure. The keys in the fish plate are so short longitudinally with

respect to the chord, that if subjected to the working strength of the other parts of the joint, they will either rotate or approach

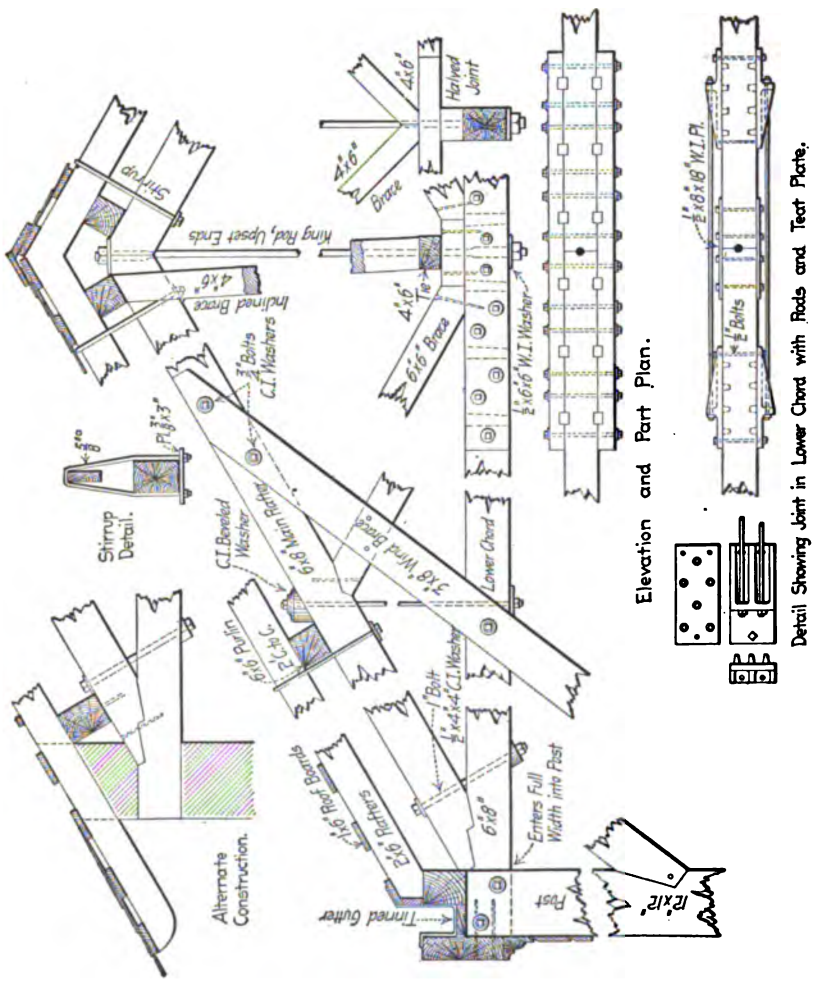


FIG. 69e.

so near to this condition as to lack a fair degree of security. The keys receive bearing on the sides of their fibers, thus lowering the allowable unit-stress in compression. Since they also

tend to shear in a plane parallel to the fibers, but in a direction transverse to the fibers, a physical property of wood is involved, concerning which no published results of tests are available.

In regard to the splice with cast-iron plates connected by rods, it may be questioned whether the conical projections furnish sufficient bearing area according to the principles given in Art. 23. Where a number of bearings are to act in harmony, the holes should be bored with the aid of a templet which corresponds exactly with the casting. In the end joint the second step would be more effective if the longer face of the triangular

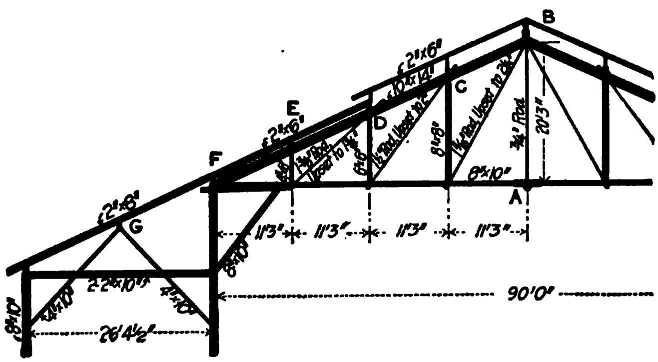


FIG. 69*f*. Outline of Main Truss, Forestry Building.

notch coincided with the lower surface of the upper chord timber. The proposed modification either increases the length of shearing surface between the notches, or places the two shearing surfaces at different heights, depending upon the relative depths of bearing adopted (Art. 31). The sections of the purlins are not well proportioned to develop their strength as beams. The so-called stirrup used to hold down the rafter is an expensive fastening, for which spikes and a wooden block may be substituted.

Fig. 69*f* gives an outline diagram of one of the main roof trusses of the Forestry building, and Fig. 69*g* shows the kind

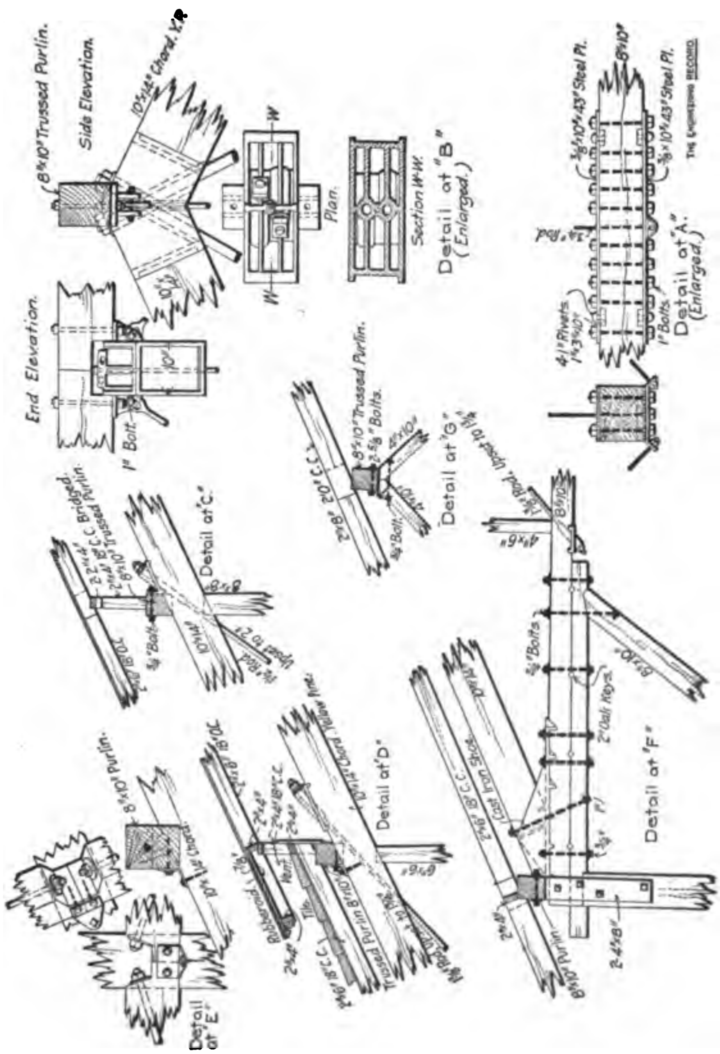


FIG. 69g. Details of Roof Truss of Forestry Building, Pan-American Exposition.

of details which are apparently necessary when an attempt is made to construct chiefly in wood a type of roof truss which is primarily adapted for wrought-iron or steel. An unusually long cast-iron shoe receives the bearing of the upper chord, and the casting at the peak is very elaborate. As only a single diagonal rod is used on each side of that joint, there is a rotating moment developed which produces secondary bending stresses in the chord timbers. The three beveled washers shown are of the type illustrated in Fig *20*, except that they are single instead of double. The splice in the lower chord is a combination of the tabled and plain fish-plate joint (see Art. 27). It should be especially noted that the trussed purlins are placed with their sides horizontal and vertical (see Art. 57). The lower chord and bolster are connected to the supporting post by a plaster joint. The details also show the upper end of the knee brace connecting the roof truss and post, and the relation to its joint with the lower chord to the adjacent panel point of the truss.

Prob. 69. Consult an article on The Evolution of Structural Design by F. T. LLEWELLYN in Journal Association of Engineering Societies, vol. 21, page 173, November, 1898; and notice what elements and conditions have governed the development of the form and details of wooden and combination roof trusses.

ART. 70. EXAMPLES FROM PRACTICE.

Fig. *70a* gives the elevation of the half span of one of the main roof trusses for the Mess Hall and Kitchen of the War College and Engineer Post at Washington, D.C. It was designed in 1903 by McKIM, MEAD & WHITE. The effective span is 43 feet 8 inches, the depth center to center of chords is 9 feet 7 inches, and the trusses are spaced 11 feet 11 inches between centers. The sheathing of plank is supported directly by the trusses, and the slate roofing is laid upon that. The trusses were designed for a total load of 60 pounds per square foot, one-

half of which is an allowance for both snow and wind loads. The details are clearly shown in the illustration.

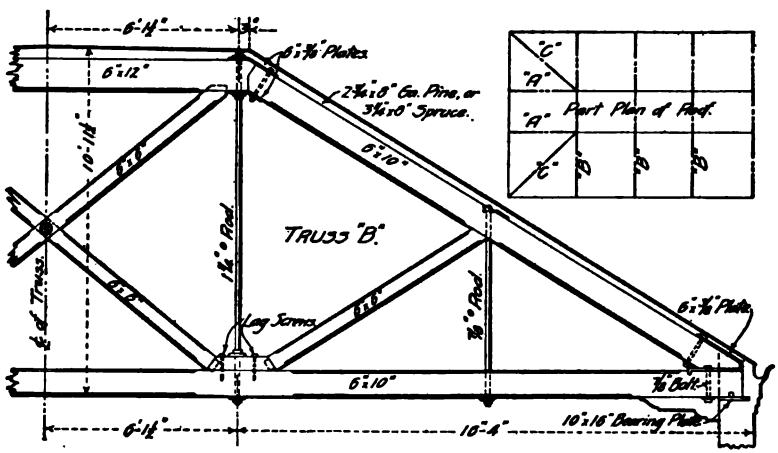


FIG. 70*a*. Roof Truss for Mess Hall and Kitchen of War College and Engineer Post, Washington, D.C.

Fig. 70*b* shows the elevation of one-half span of a roof truss and a half cross-section of the freight warehouse at Atlanta, Ga., of the Central of Georgia Railway. The structure was designed in 1903. The roof trusses are very simple in design, consisting entirely of wood and connected at the panel points by bolts. Since the braces consist of single sticks and are held between the chord timbers in the manner shown, the center lines of truss members do not intersect in a point at the panel points. This involves secondary bending moments which are taken into account in designing the section areas of the chords. The span, depth, and composition of the truss members are given on the drawing.

Fig. 70*c* gives the elevation of the half span of one of the main roof trusses of the north wing of Goldwin Smith Hall at Cornell University. This portion of the building was originally built for the Dairy Department of the College of Agriculture

and was designed in 1893. The span and rise of the truss are given on the drawing. The trusses are spaced 8 feet apart, and directly support the heavy plank sheathing which was laid diagonally over the larger portion of the roof to secure more rigid

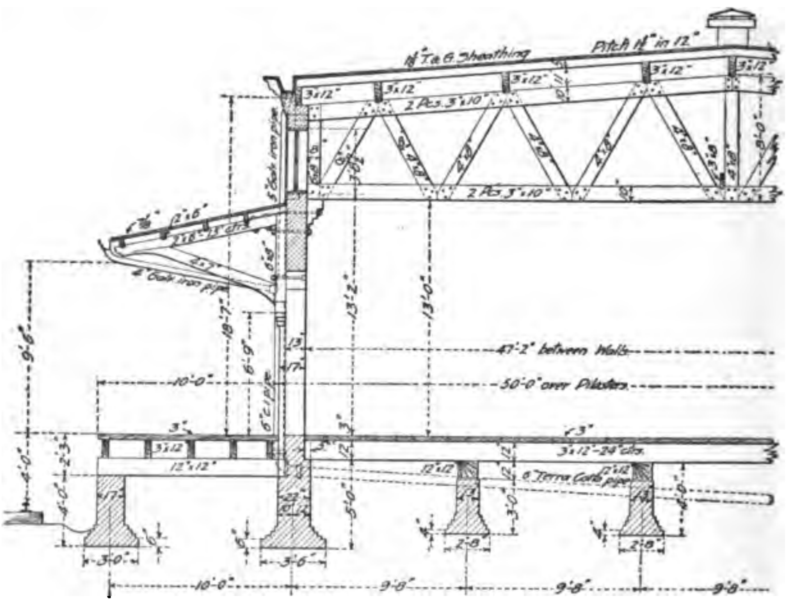


FIG. 70*b*. Cross-section of Freight Warehouse of Central of Georgia Railway at Atlanta, Ga.

bracing. The roof covering consists of a heavy glazed interlocking tile. The details of the pin-connected joint are shown on an enlarged scale as well as those of the bent bearing plate at the end joint of the truss. The strut enters the socket of a casting at the lower end which takes bearing on the pin.

The upper drawing on Plate V shows a roof truss which is built mainly of yellow pine, steel being used for a few of the web tension members and for splices and connection bolts. The trusses divide the roof into three bays of about 22 feet and two of 27 feet, and are proportioned for the standard roof loads pre-

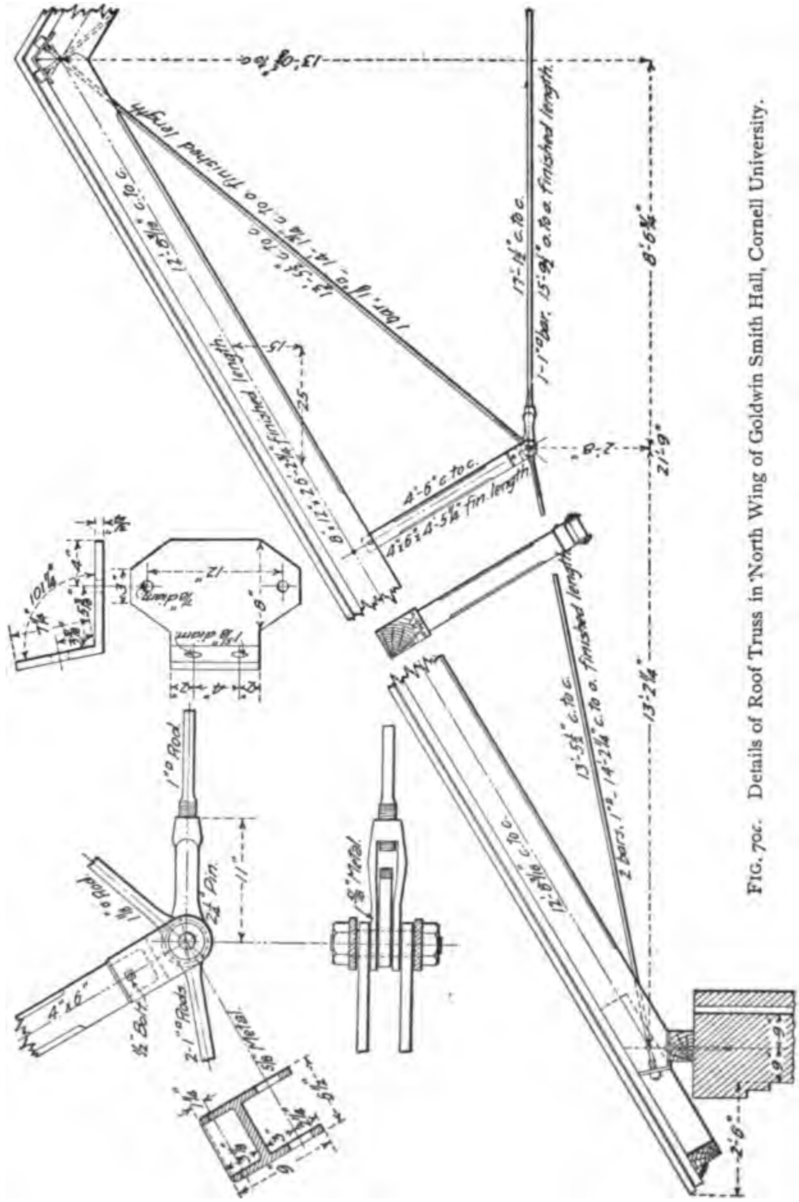


FIG. 70c. Details of Roof Truss in North Wing of Goldwin Smith Hall, Cornell University.

scribed by the building laws. The trusses supporting the longer bays differ from that shown only in having slightly larger cross-sections in some of the members.

The top chord timbers all reach from the end of the truss to the center and make a simple miter-joint there under a ribbed cast-iron saddle which is beveled to fit over the ridge, but is not bolted to it. The vertical center rod takes bearing on this casting, and its upper end is received in a hole mortised in the ridge purlin, which is secured there by a wood screw through the top flange of the saddle.

The bottom chord is made of four sticks, two of them 43 feet 10 inches long, and two 23 feet 6 inches long, arranged to break joints 10 feet 2 inches from the center. Each splice consists of a tabled fish-plate joint with steel plates. The rivet heads on the outside of the plates are not countersunk. A wooden filler is put between the chord pieces at the splices and there are two rows of bolts through both the continuous timber and the splice, arranged so that there is a pair of bolts just beyond each bearing strip counting from the splice. It should be noted that in practice the bolts are not placed in their proper position to resist the rotating moment as explained in Art. 21.

In the middle of the truss, the web members have shoulders bearing on shallow seats mortised in the inner horizontal faces of the chords, over the whole area of intersection, and they have beveled tenons extending beyond the seats to the outside of the chords and abutting with the adjacent web member, being secured by two through bolts. At the ends of the trusses, the larger compressive stresses are resisted by inclined posts with square ends set on oak angle blocks notched $1\frac{3}{4}$ inches into the chords across the face. The corresponding tensile stresses are taken by single round screw rods having nuts bearing on cast-iron angle blocks or beveled washers engaging notches on the outside of the chords. All of them are the width of the chord,

but their lengths and the depths of the toe pieces are proportioned to the stresses.

The end bearings are in bolsters 6 feet long, which are keyed to the lower chords and were intended to be stiffened by knee braces, but the latter were omitted because they interfered with the required clearance. The design was especially intended to secure an efficient disposition of material, simple framing, economical construction, to develop the full strength of the members in the connections and splices and to have the separate pieces so secured that they will not be loosened nor permit deformation to result from shrinkage of the timber. The design was made by G. H. CHAMBERLAIN and BERNT BERGER, the architect and consulting engineer respectively. This description and the illustration is reprinted from Engineering Record, vol. 42, page 155, Aug. 18, 1900.

The lower drawing on Plate V shows the cross-section of the Central Avenue freight station at Cincinnati, Ohio, of the Cleveland, Cincinnati, Chicago, and St. Louis Railway, built in 1900 to replace the old structure which was destroyed by fire. Most of the general dimensions and a number of details are given on the drawing, the latter to an enlarged scale. The main columns of the building are 21 feet apart longitudinally and carry wall plates 8 by 12 inches for the support of the platform roofs, and above these the columns are carried up to support the wall plates of the central roof, the trusses of which are spaced 21 feet between centers. The load is not carried entirely on the wall plates, but is transferred by struts and braces to bearing points on the columns.

The combination roof truss has all its tension members composed of square bars. At the intermediate panel points of the lower chord the rods are looped over $2\frac{1}{2}$-inch turned pins, and at the end panel points over $1\frac{1}{2}$-inch pins. The upper ends of the inclined tension members take bearing on the casting at the peak by nuts and washers. The details of the three castings are shown.

The roof over the inbound platform has 12 by 16-inch rafter beams following the slope of the tar and gravel roof, which rest on bolsters supported by the main posts and by smaller posts along the edge of the platform. Between the beams are purlins supported by double stirrups, on which the $2\frac{1}{2}$-inch sheathing is laid. The roof over the outbound platform has rafter beams 8 by 8 inches in section, supported on the wall plates over the line of main columns and on iron columns on the platform, and they extend 7 feet beyond to support the overhanging roof. Small purlins spaced close together rest on top of the beams, and in turn support $\frac{7}{8}$-inch sheathing. There is a system of angle braces throughout, and special attention is called to the use of a straining beam under the rafter beams of the outbound platform, and to the straining beam between the angle brace and the nearest strut of the roof truss. The illustration and data regarding this roof truss are taken from an article in Engineering News, vol. 46, page 16, July 11, 1901.

ART. 71. REFERENCES TO ENGINEERING LITERATURE.

The following references relate only to roof trusses. No attempt is made to refer to all the engineering periodicals published in this country, nor to include every article on the subject to be found in the periodicals selected. The aim has been merely to give a sufficient number of selected references, to enable the student or young engineer to notice the types of roof trusses in use in this country and to study the details employed in various designs. Card catalogues of references, arranged in accordance with a definite system of classification, form a part of the equipment of first-class offices of engineers and architects.

WOODEN AND COMBINATION ROOF TRUSSES.

Modern Industrial Plant.—Eng. Rec., v. 23, p. 356, May 2, 1891.

Jersey City Freight Terminus; Lehigh Valley Railroad.—R. R. Gaz., v. 23, p. 610 (inset), Sept. 4, 1891.

Tacoma Shops of the Northern Pacific Railroad; Combination Trusses of Large Span. — Eng. News, v. 27, p. 170 (inset), Feb. 20, 1892.

Buildings and Structures of American Railroads; No. 16, Engine Houses, by Walter G. Berg. — R. R. Gaz., v. 24, p. 203, March 18, 1892.

World's Columbian Exposition; Transportation Building. — Eng. News, v. 28, p. 605 (inset), Dec. 29, 1892.

Buildings and Structures of American Railroads, by Walter G. Berg, 1892.

World's Columbian Exposition; Electricity Building. — Eng. News, v. 29, p. 434 (inset), May 11, 1893.

Burnside Shops, Illinois Central R. R. — Eng. News, v. 35, p. 402, June 18, 1896; v. 36, p. 34 (inset), July 16, 1896.

Sanger Hall, Philadelphia; Design and Construction of a Large Temporary Auditorium. — Eng. Rec., v. 35, p. 120, Jan. 9, 1897.

New Freight Station at Columbus, O., for the Pittsburgh, Cincinnati, Chicago, and St. Louis Ry. — Eng. News, v. 37, p. 66 (inset), Feb. 4, 1897.

New Passenger and Freight Station at Montgomery, Ala.; L. & N. R. R. — Eng. News, v. 38, p. 114 (inset), Aug. 19, 1897.

New Shops of the Peoria & Eastern at Urbana. — R. R. Gaz., v. 29, p. 666, Sept. 24, 1897.

New Boston & Maine Shops at Concord. — R. R. Gaz., v. 30, p. 76 (inset), Feb. 4, 1898.

Corlears Hook Park Overlook. — Eng. Rec., v. 37, p. 235, Feb. 12, 1898.

New Foundry of the Semi-Steel Co. — Eng. News, v. 39, p. 405, June 23, 1898.

An Unusual Framed Building. — Eng. Rec., v. 40, p. 508, Nov. 18, 1899.

Scissors Truss, by F. E. Kidder. — Eng. Rec., v. 40, p. 440, Oct. 7, 1899; by W. C. West, v. 40, p. 488, Oct. 21, 1899; by F. E. Kidder, v. 40, p. 710, Dec. 23, 1899.

Thackeray Garbage Furnaces at San Francisco, Cal., by F. J. Mills. — Eng. News, v. 43, p. 318, May 17, 1900.

Special School House Roof Truss. — Eng. Rec., v. 42, p. 257, Sept. 15, 1900.

Ethnology Building at the Pan-American Exposition. — Eng. Rec., v. 44, p. 226, Sept. 7, 1901.

East Orange Town Hall Roof. — Eng. Rec., v. 44, p. 403, Oct. 26, 1901.

Memphis Shops of the Illinois Central. — R. R. Gaz., v. 34, p. 792, Oct. 17, 1902.

Bi-Centennial Memorial Building of Yale University. — Eng. Rec., v. 47, p. 604, June 6, 1903.

Details of a Large Timber Roof of a Public Shelter in Van Cortlandt Park, New York. — Eng. Rec., v. 49, p. 259, Feb. 27, 1904.

New B. & O. Freight House at Columbus. — R. R. Gaz., v. 38, p. 403, April 28, 1905.

New Freight Station at Cincinnati, O.; Cincinnati Southern Ry. — Eng. News, v. 54, p. 593, Dec. 7, 1905.

Difficult Reconstruction of a Church Roof. — Eng. Rec., v. 53, p. 428, March 31, 1906.

Saw-tooth Roofs for Factories, by Knight C. Richmond. — Eng. News, v. 56, p. 627, Dec. 13, 1906.

Large Wooden Frame Shop. — Eng. Rec., v. 57, p. 494, April 11, 1908.

170-Ft. Wooden Roof Truss. — Eng. Rec., v. 55, p. 348, March 16, 1907.

Long Span Timber Roof Truss, by J. C. Lathrop. — Eng. News, v. 59, p. 628, June 11, 1908.

Standard Boiler House Design of the Oliver Iron Mining Co., by A. M. Gow. — Eng. News, v. 60, p. 294, Sept. 17, 1908.

Large Concrete and Timber Coal Pocket. — Eng. Rec., v. 59, p. 603, May 8, 1909.

CHAPTER V.

EXAMPLES OF FRAMING IN PRACTICE.

ART. 72. SLOW-BURNING CONSTRUCTION.

Slow-burning or mill construction, as defined by EDWARD ATKINSON, its most noted exponent, consists in so disposing the timber and plank in heavy solid masses as to expose the least number of corners or ignitable projections to fire, to the end also that when fire occurs it may be most readily reached by water from sprinklers or hose. It consists in separating every floor from every other floor by incombustible stops, by automatic hatchways, by encasing stairways and belts either in brick or incombustible partitions, so that a fire shall be retarded in passing from floor to floor to the utmost that is consistent with the use of wood or any material in construction that is not absolutely fireproof. It also consists in guarding the ceilings over all specially hazardous stock or processes with fire-retardent material . . . and in so constructing the mill, workshop, or warehouse that fire shall pass as slowly as possible from one part of the building to another, but also in providing all suitable safeguards against fire.

A valuable illustrated description of this method of construction, and containing general directions for designers, is published as Report No. 5 of the Insurance Engineering Experiment Station under the direction of the Boston Manufacturers Mutual Fire Insurance Company. It contains nine folding plates of plans which illustrate good practice and gives directions regarding details based upon extensive experience. The pamphlet also contains a table of weights of merchandise giving the approximate

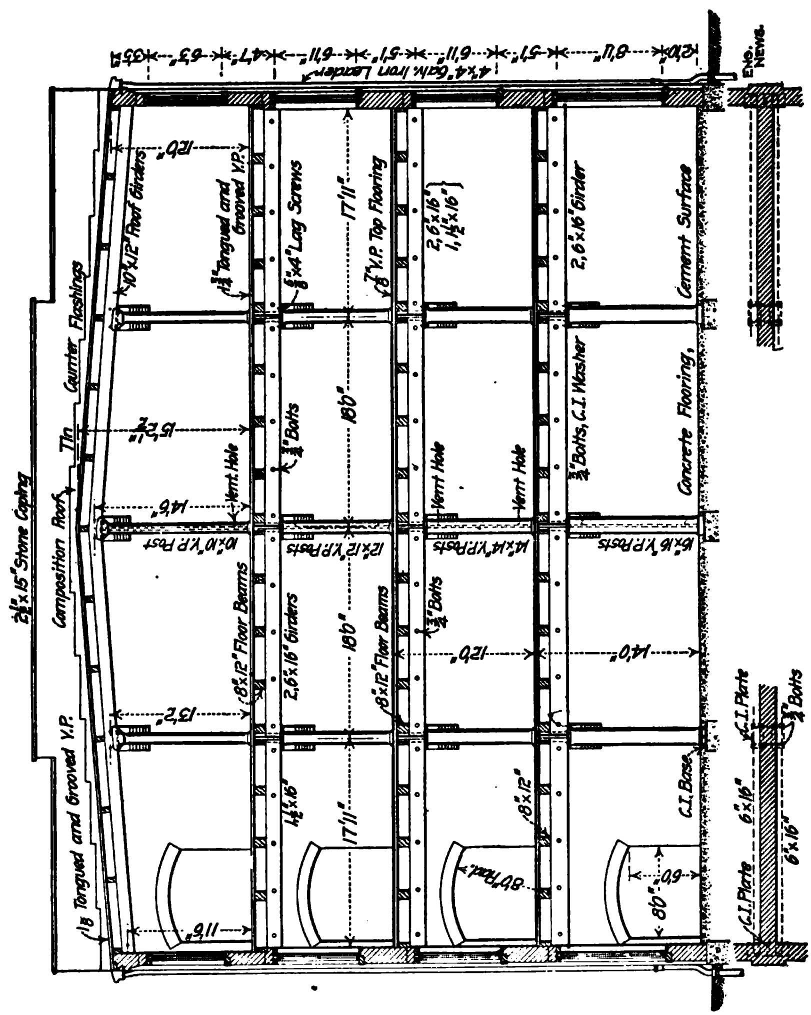


FIG. 724. Transverse Section of Pattern Building, New Worthington Hydraulic Works.

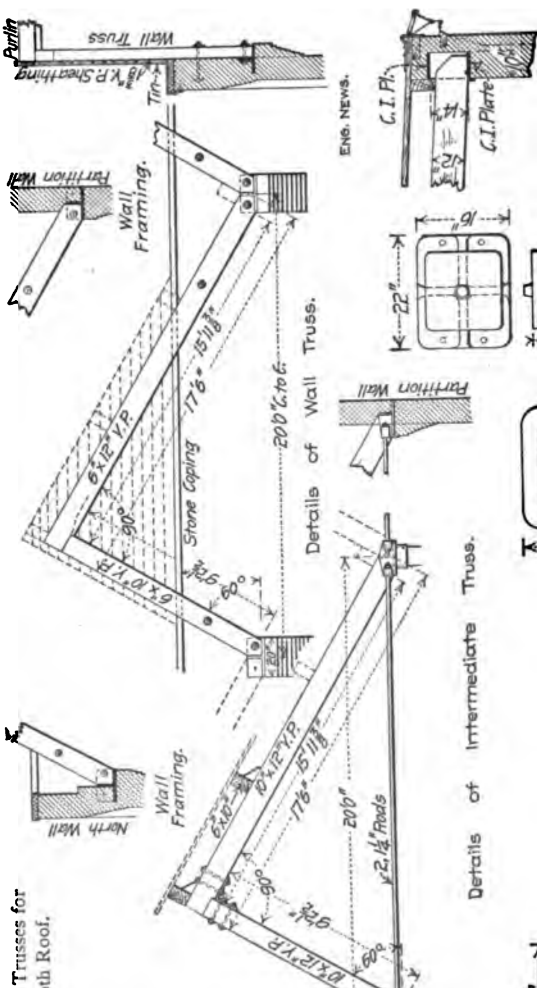
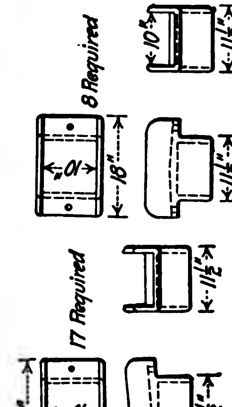


FIG. 72b. Trusses for Saw-tooth Roof.



FIGS. 72c and d. Post Caps under Rafters.

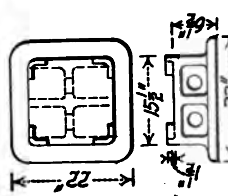


FIG. 72e. Base Plate.

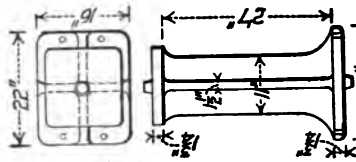


FIG. 72f. Post Cap.

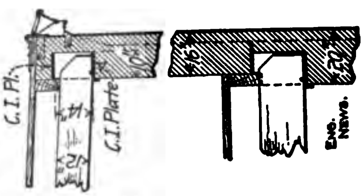


FIG. 72g. Beam Anchorage.

dimensions of the packages, their cubic contents, and floor spaces required, as well as the weights per square foot and per cubic foot.

The standard mill is planned with heavy beams from 8 to 11 feet between centers, and supported by posts composed of single sticks of square cross-section. In determining the sizes of beams the three elements of weight, deflection, and vibration are considered. A heavy plank floor is laid on the beams, and on this a single thickness of hard close-grained floor boards with two layers of resin-sized paper between. In the best mills a board floor is laid diagonally or at right angles to the plank, and over that a top floor of birch or maple laid lengthwise. This intermediate floor gives great resistance to lateral vibration, and can be ordinarily of the cheapest lumber obtainable, of practically uniform thickness, and is well worth the additional cost. In some cases the flooring consists of two-inch joists on edge spiked together closely side by side, the thickness of the floor varying with the loads and span from 5 to 8 inches or more.

Figs. *72a* and *b* show some details of modern slow-burning construction, as used in the pattern building of the New Worthington Hydraulic Works at Harrison, N.J. The building, which is 552 feet long, is separated into four divisions by brick partition walls. The section shown is taken close to the middle wall. The first division is occupied by the office and drawing room, the second is the pattern shop proper, and the other two are used as pattern houses. The longitudinal spacing of the columns in these divisions are 20, 15, and 10 feet respectively.

The post caps are cruciform pedestals of cast-iron of the general form and construction indicated by Fig. *72f*. These caps are inserted between stories and their purpose is to give the floor beams a connection with the columns and at the same time to make the column continuous from floor to floor. Where the post takes the rafter, use is made of the special caps shown in Figs. *72c* and *d*, and at the bottoms of the columns are used base plates

constructed as shown by Fig. *72e*. Details of the wall connections for the girders and rafters are shown by Fig. *72g*.

Over the office portion of the building the peaked roof shown by Fig. *72a* is replaced by a saw-tooth roof to give light to the drawing room on the top floor. There are five bays of this roof, each bay running transversely across the building and consisting of two wall trusses and three intermediate trusses carrying purlins and roofing. The drawings of Fig. *72b* show these trusses and their connections. See Engineering News, vol. 50, page 584, Dec. 31, 1903.

A number of special details used in slow-burning construction have been described in previous chapters, including pintles and post caps, beam hangers, beam anchors, etc., accompanied by notes on their relation to fire hazard. For additional information see the references given in Art. 77.

ART. 73. TRESTLE CONSTRUCTION.

Various methods of framing the bents of wooden trestles are given in connection with the treatment of bolts and nuts in Art. 1; of drift bolts in Art. 9; of dowels in Art. 14; of metal straps and plates in Art. 18; of mortise-and-tenon joints in Art. 30; of wooden beams in Art. 38; of wooden posts in Art. 50; and of bolsters in Art. 51. The arrangement of the stringers is described in Art. 39 on packed stringers. The relative advantages and disadvantages of the different methods of framing used in practice have been discussed in engineering periodicals and at conventions of the American Railway Superintendents of Bridges and Buildings. Several selected references on this subject are given in Art. 77.

The sizes of timbers in trestle bents are determined chiefly by practical considerations apart from those required for the strength of the structure. For instance, the posts usually have a far greater degree of security than the caps, but since timbers

have to be kept in stock and shipped frequently on telegraphic orders to replace an old structure which was suddenly destroyed by fire or flood, it is convenient to frame different members from sticks of the same size.

In Bulletin No. 12 of the U. S. Division of Forestry, published in 1896, on Economical Designing of Timber Trestle Bridges by A. L. JOHNSON, tables are given regarding the practice on 15 American railroads at that time. It was found that 11 species of wood were used for piles, 6 for stringers, 10 for caps, and 9 for posts. An analysis was also made of the degrees of security respectively for the flexural strength of stringers, for the posts as columns, and for the bearing on the side of the fibers under stringers, caps, and posts. The results indicate that bearing apparently failed to receive sufficient attention in designing.

Since the relative strength of the posts is so much greater than that of the caps and stringers, it is possible to use double posts, caps, and sills without any sacrifice in the strength of the structure as a whole. The size of the post is determined by its bearing on the cap and sill, and the bearing area is the same whether one or two sticks are used, provided the combined section area is the same. In Bulletin No. 12 the author proposed to use different species of timber in order to secure a more economical design and at the same time more nearly uniform degrees of security in the members. On account of its greater resistance to compression on the side of the fiber, white oak was recommended for caps and sills, longleaf yellow pine for stringers, and red cypress for posts. Comparative estimates of cost were given showing a saving in favor of the plans proposed.

The latter part of the bulletin contains a review of the paper by G. LINDENTHAL, and some notes by WALTER G. BERG, discussing the practical objections to the changes proposed, and the different conditions which relate to true economy in trestle

construction and maintenance. The most extensive treatment of American trestle construction is given in a Treatise on Wooden Trestle Bridges, by WOLCOTT C. FOSTER, which contains 48 standard plans of pile and frame trestles. The standards adopted by the railroads are revised at intervals. See references in Art. 77.

It is of great importance to prevent hot cinders from lodging on the structure which will set it on fire. The cinders will either remain on the stringers between the ties, or on the cap between the stringers. The most efficient protection of trestles against fire is illustrated in Fig. 73*a*, where galvanized sheet iron is laid

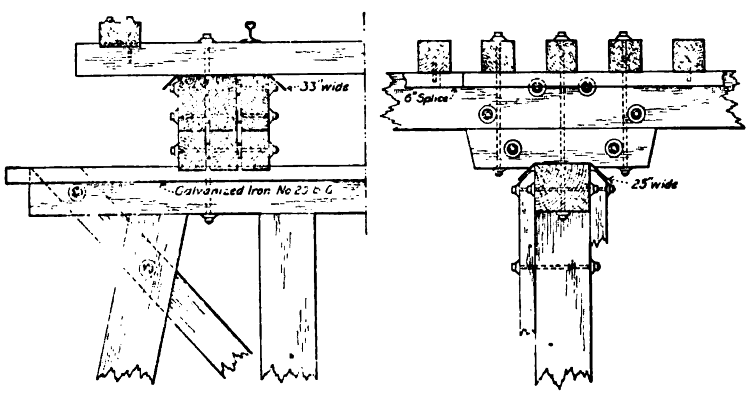


FIG. 73*a*. Iron Covering on Stringers and Cap to prevent Fire and Decay.

on both stringers and cap and bent down over the edges. The sheets should be about 11 inches wider than the timbers covered, and of about 22 to 27 gage in thickness, having a lap longitudinally of about 6 inches. They should be protected from rust by heating the iron and painting it with hot tar. The iron dowels connecting the ties to the stringers will hold the sheet iron in place.

The sheet iron also performs another important function in protecting the tops of stringers and caps from water and thereby materially prolonging the life of these timbers. Since the

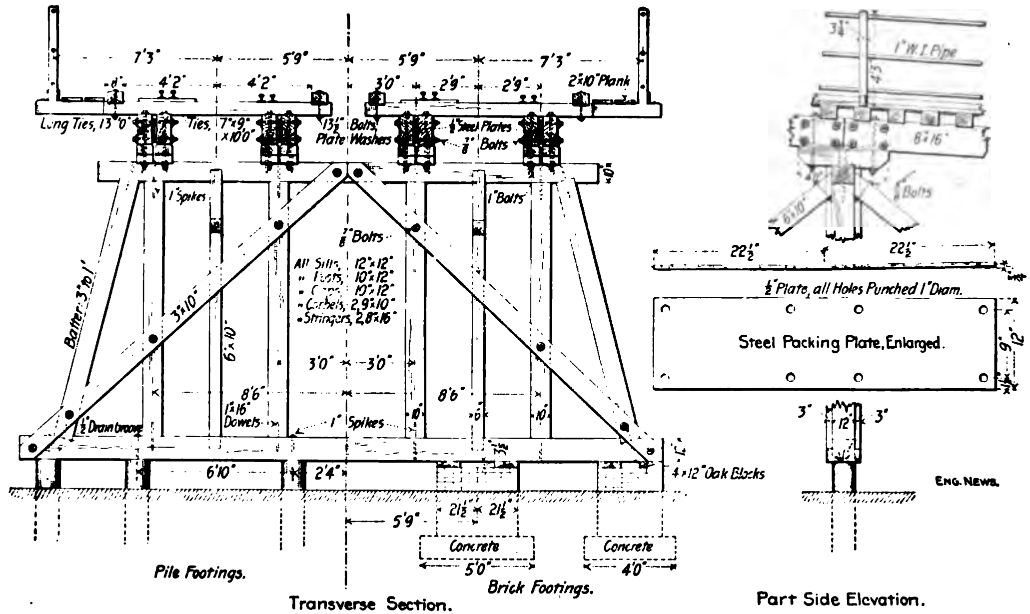


FIG. 73*b*. Details of Bent used in Trestle at Long Branch, N J., for the Atlantic Highlands, Red Bank, and Long Branch Electric Railway.

Noticeable features are the use of square ends for posts held in place by dowels and the entire absence of mortise-and-tenon joints. The ends of the posts and the triangular notches cut for them were thoroughly coated with asphaltum. At alternate bents steel packing plates are used instead of wooden ones and are separated from the stringers by washers; and at the other bents plate spacers 12 inches long are connected in a similar manner. See Engineering News, vol. 39, page 21, Jan. 13, 1898.

FIG. 73c. Stairway Approach to Rhawn Street Viaduct, Philadelphia, 1900.

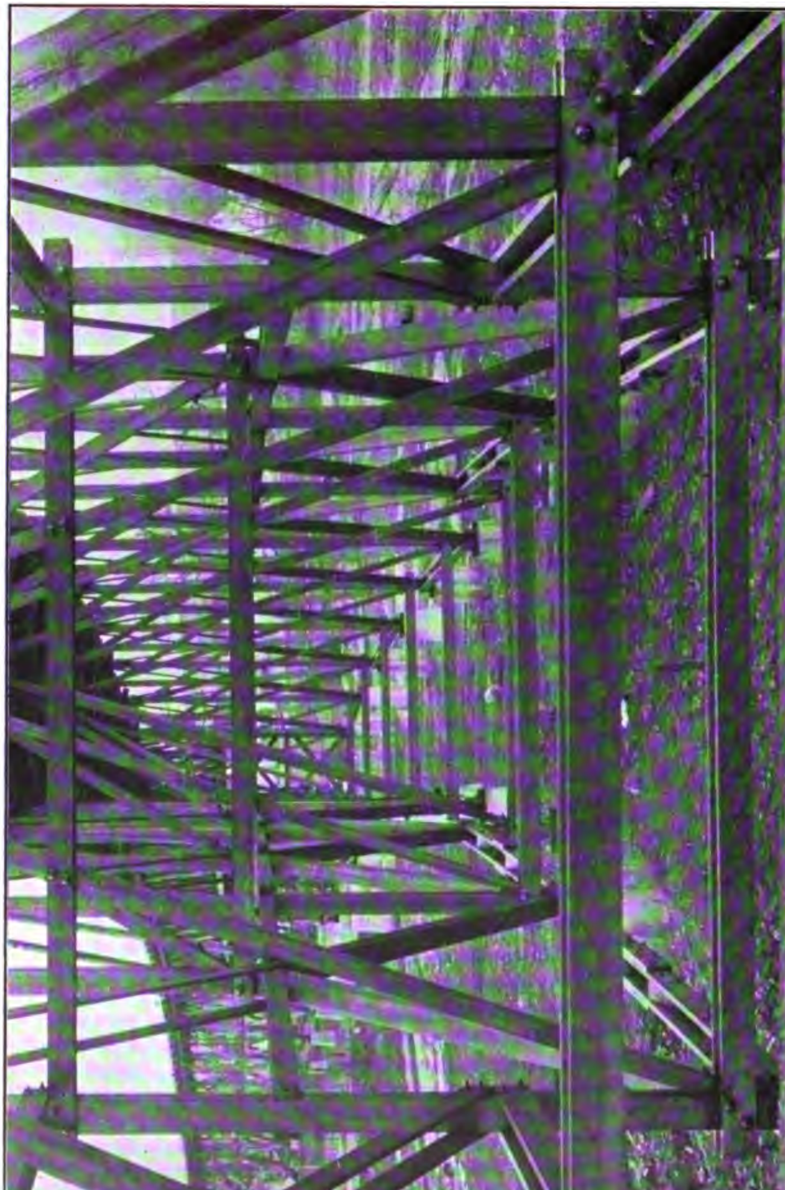


FIG. 73*d*. Highway Viaduct on Rhawn Street, Philadelphia, Pa., 1900.

strength of stringers is impaired to some extent by preservative processes the importance of protecting them from decay by covers deserves increasing consideration.

Fig. 73*c* shows the braced towers of the stairway approach to the Rhawn Street timber viaduct in Philadelphia, which crosses Pennypack Creek. There are two viaducts respectively 529 and 942 feet long. As the structures are of wood and may require replacement, the utmost economy in the design was exercised to diminish the first cost, and at the same time to meet all the requirements of traffic. The wood is yellow pine and the bracing is mainly bolted in place. The lower timbers in the side diagonals and the diagonals of the transverse bracing are connected to the post by single step joints. Fig. 73*d* also shows the splices in the bottom longitudinal braces in which tabled fish plates are employed.

ART. 74. SMALL BRIDGE TRUSSES.

Figs. 74*a* and *b* show the details of a temporary highway bridge erected by the Rutland-Canadian Railway on its new line between Burlington and Alburg across Lake Champlain, pend-

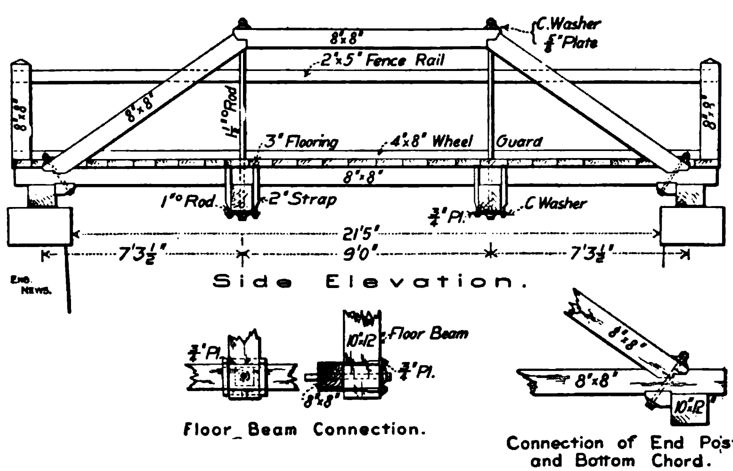
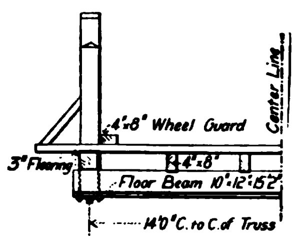


FIG. 74*a*. Temporary Highway Bridge.

ing the delivery of the steel bridges. The details are clearly indicated on the drawings and require no explanation. The illustration is reprinted from an article on The Rutland-Canadian Railway and its Structures, in Engineering News, vol. 49, page 46, Jan. 15, 1903.

Fig. 74c gives a side view of a short span single track railroad bridge in the Rocky Mountain region of British Columbia. It crosses the north branch of Michel Creek between the stations McGillivray and Michel on the Crow's Nest Line of the Canadian Pacific Railway. Cast-iron angle blocks are used at the intermediate joints of the upper and lower chords, while the single step joint like Fig. 31*a* is used at both ends of the inclined end posts. The sticks in both chords are continuous. A bearing block for the diagonal strut is inserted at the end joint of each upper chord. As the cross-ties rest on the lower chord, the chord members are subject to combined tension and flexure.



Half End Elevation.

FIG. 74*b*.

The Municipal Ferry transfer bridges shown in Fig. 74*d* are built entirely of wood except for the bolts and a few tie rods used for connections. They are made unusually heavy and have four arch ribs or quasi-trusses of the bowstring form to support the platform. The lower chords are heavy continuous timbers, and on account of their flexural strength no diagonals are inserted in the trusses. The wooden floor is supported on two tiers of heavy longitudinal and transverse timbers spaced close together, bolted at every intersection and suspended at seven panel points from each arched chord by a pair of screw rods.

The river end of the floor is curved exactly to fit the end of the ferry boat, and is faced with vertical 4-inch oak lagging and ½-inch iron bands spiked to horizontal oak saddle pieces. These

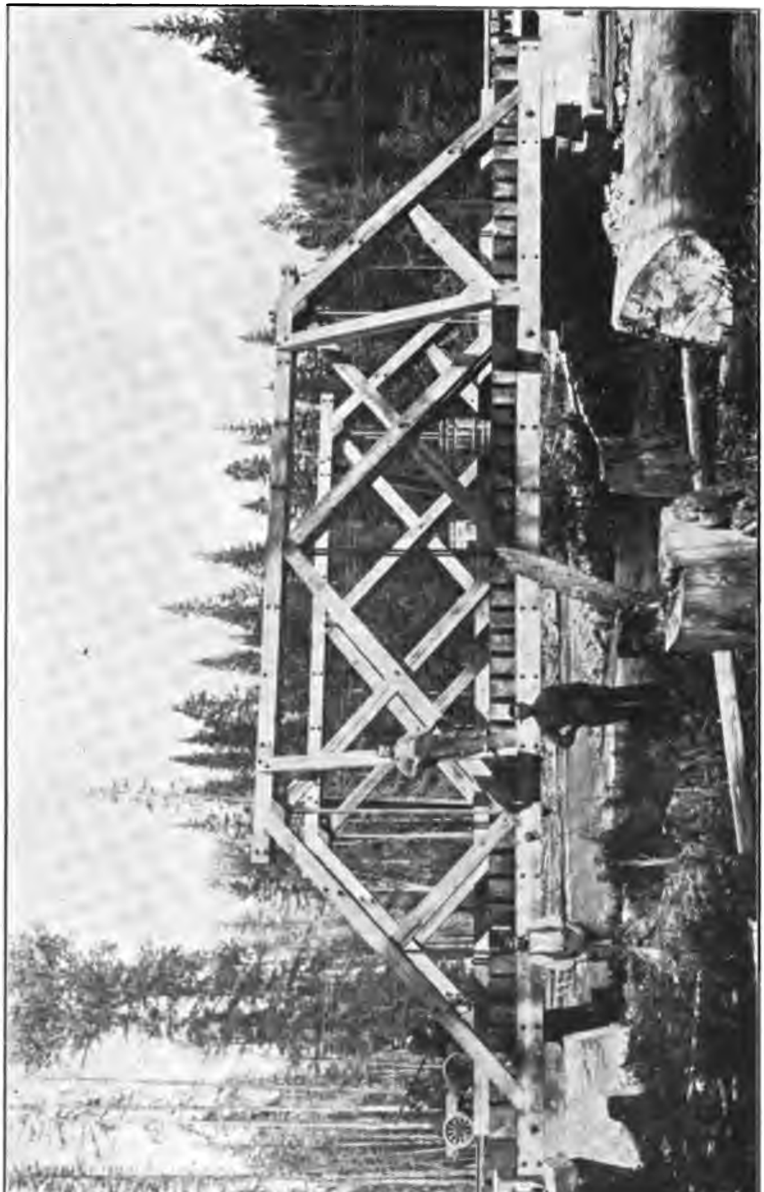


FIG. 74c. Pony Howe Truss Bridge on Crow's Nest Line, Canadian Pacific Railway, 1901. Span 50 feet.

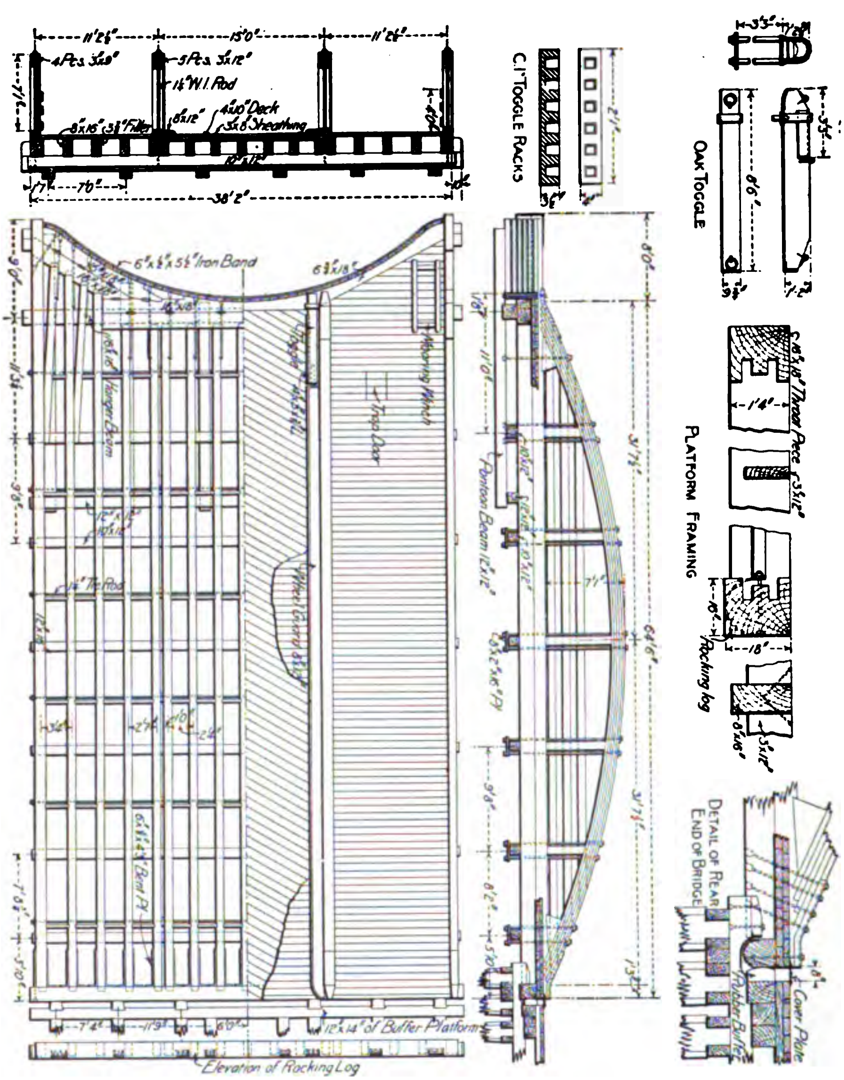


FIG. 74*d*. Framing of Transfer Bridge at Municipal Ferry House near Battery Park, New York City. Built in 1907.

have almost continuous backing, with longitudinal and inclined struts bolted to the main girders in the floor, so as to distribute thoroughly any impact received. The curved chords are built up of 3-inch planks with lapped joints, which are bolted together with $\frac{3}{4}$-inch bolts spaced about 12 inches apart. Their ends are stepped and bolted to the 8 by 16-inch bottom chord timbers. The 16 by 16-inch transverse oak timber at the shore end of the platform is thoroughly braced and connected to the longitudinal timbers and upper chords, and has a cylindrical lower surface faced with a steel plate, which revolves in a corresponding steel-faced quoin fixed to the tops of the platform piles.

The outer end of the bridge is supported on a pontoon, and is automatically adjusted by the tide itself, except for slight dif-

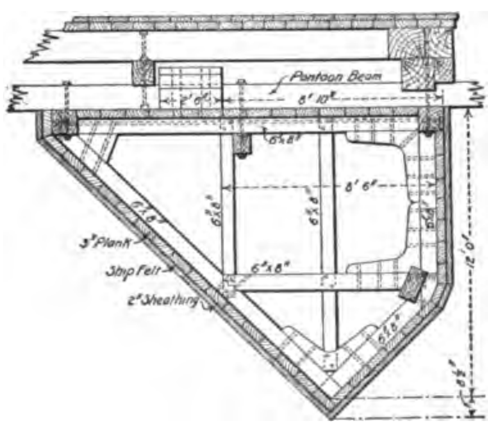


FIG. 74*e*. Section of Transfer Bridge Pontoon.

ferences in elevation, which are controlled by windlasses. The pontoons are built entirely of wood and extend nearly across the full width of the bridge, which takes bearing on them with each of its six principal longitudinal beams, and are thoroughly bolted through its principal deck members. A section of the pontoon is given in Fig. 74*e*. The transverse vertical frames, 27 inches apart, are made with 6 by 8-inch lap-jointed yellow pine timbers,

with hackmatack knees and 1¼-inch treenails at their intersections. They are covered with an inner longitudinal planking of 3-inch yellow pine and an outer transverse planking of 2-inch spruce, both caulked and separated by a layer of ship felt. The top is covered with a tight deck of 4-inch planks. The framing of the piers, ferry racks, etc., is described and illustrated in an article in Engineering News, vol. 58, page 525, Nov. 7, 1908, from which the information on the ferry bridge and pontoon is taken.

The temporary arched bridges of short span over canals and waterways at the Pan-American Exposition were constructed on the site by carpenters and laborers using ordinary commercial sizes and moderate lengths of pine and hemlock timber, mostly 4 by 12 inches or less in cross-section, bolted together without special forgings or any but a very few of the simplest castings. The simple character of the details is indicated in Fig. 74*f*. The conditions required considerable clearance for pleasure boats, to keep the roadway grade as low as possible, and to be safe and solid under a heavy live load of pedestrians.

The principal framework consisted of nine arched ribs, spaced 6 to 6½ feet between centers, and having solid spandrel bracing extending to the longitudinal stringers, which are nearly parallel to the grade of the roadway. The ribs thus resembled pairs of solid triangular girders continuous over the crown, and having solid end bearings, with long rigid connections with the vertical abutment piles. The ribs were connected together by the floor joists and floor planks, and at the intrados by horizontal transverse struts and the sheathing which covered them. This construction shows that the ribs did not act as true arches, but as containing a combination of elements consisting of a beam with fixed ends, a cantilever, and an arch. Each rib was composed of four planks bolted together and keyed with 2-inch round white oak keys. Each end of the rib took bearing on a hollow cast-

iron skewback block which was notched into a longitudinal oak cap, and this in turn mortised over one pile and connected to the other by a plaster joint. The manner in which the bridge was finished to harmonize with the architectural motives of the build-

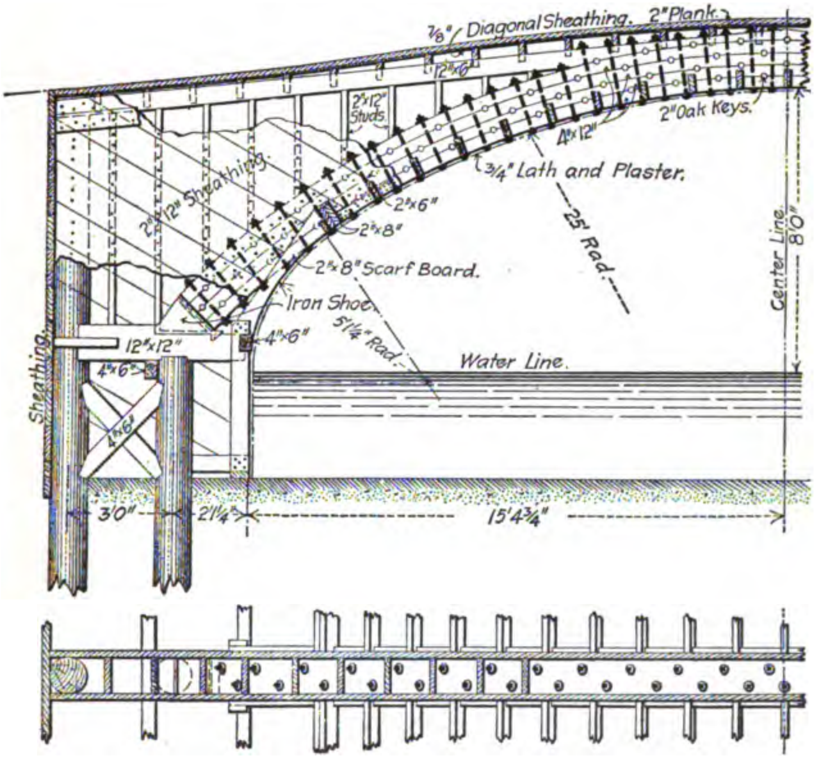


FIG. 74*f*. Wooden Bridge Construction, Pan-American Exposition.

ings and the landscape features of the vicinity are described in Engineering Record, vol. 44, page 394, Oct. 26, 1901.

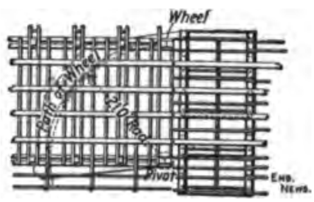
Plate VI shows the details of the swing span of a temporary wooden bridge which was built in 1905 and was in service during the construction of the steel rolling lift bridge on 22d Street, Chicago, which was completed in 1906. This span was subsequently used at Ashland Avenue and then at 35th Street, while

another one built on the same plans was in service at Archer Avenue. The first temporary swing bridge of this type was in service during the reconstruction of the Northwestern Avenue bridge, which was completed in 1904, others being used at Northwestern Avenue, North Avenue, and Kinzie Street. The design was made by the Division of Bridges and Harbor of the Bureau of Engineering of the City of Chicago.

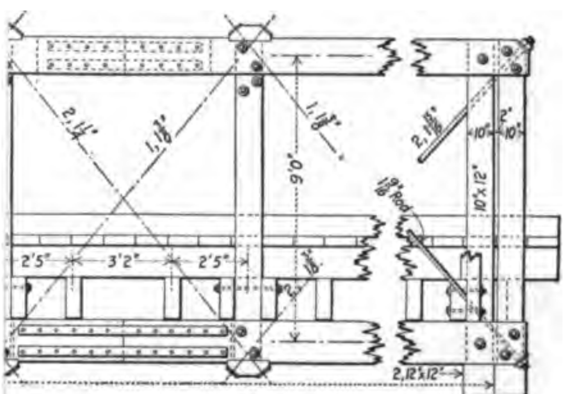
As indicated in the elevation and part plan, it has a pivot and wheel bearing respectively at the two corners of one end and is supported by a fixed bearing at the other end when closed, and by a scow or pontoon when swinging. It carried a double-track electric railway and was located a little distance up stream from the site of the permanent bridge. The clear waterway provided was 72 feet when the span was opened.

The weight of the swing span is about 100 tons, and it is designed to carry a live load of 2000 pounds per linear foot, with local wheel loads of 5000 pounds. The trusses, floor beams, and pontoon are built of yellow pine, the caps, pivot platform, and roadway flooring are of white oak, and the piles of Norway pine.

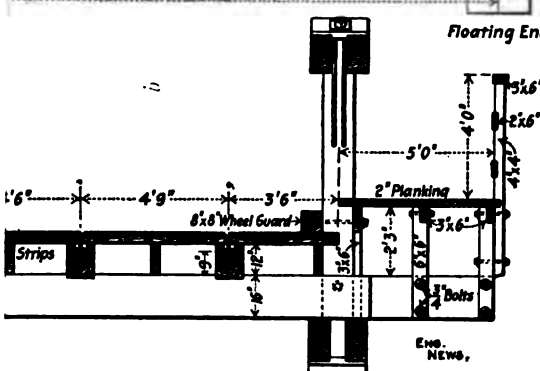
It is to be noted that the truss is of the Pratt type instead of the Howe type and that the connections for the diagonal rods are made very simple by providing bearing blocks outside of the chords. Each of the chords is composed of two lines of timber spaced a short distance apart, the rods passing between them. The ends of the verticals also pass between, while their shoulders bear against the narrow sides of the chord timbers. Each of the steel fish plates in the lower chord splices is $3\frac{1}{2}$ by $\frac{1}{2}$ inches, the filler block between the two chord timbers is 6 by 16 inches, and the bolts are 1 inch in diameter inserted in their holes with a driving fit. The fish plates on the upper chord splices are only $\frac{5}{8}$ inch thick. The remaining details of framing are indicated on the drawing and require no explanation. For



Part Plan.



Floating End.

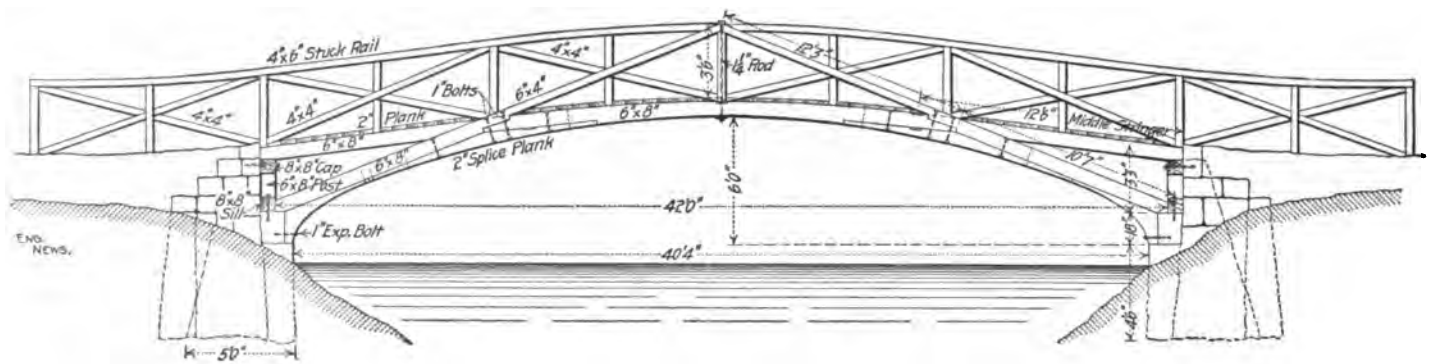


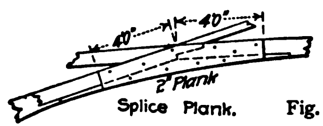
Section.

. News

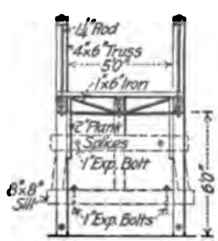
th Pontoon Swing Span over South Branch
r at Twenty-Second Street.

Built in 1905.





Splice Plank.



Cross Section at Center.

FIGS. 74*g*, *h*, and *i*. Framing of Ginn Field Foot Bridge, Metropolitan Park System.

Fig. 74*g* is an excellent illustration of careful design in framing, to secure a simple structure of pleasing lines which is well adapted to its location. It crosses the Aberjona River at the upper end of Ginn Field, a portion of the Winchester section of the Mystic Valley Parkway. In type it is a combination of a King-post truss and an arch. The middle curved portions of the arch or floor stringers were built up of 2-inch plank for economic reasons. The timber used in the structure is all kyanized spruce of extra quality, cut to exact sizes and shapes at the mill before treatment. In assembling, the ends of all timbers which butt together were given a good coat of lead, and the finished structure was given two coats of paint. The bridge was built in 1902. A classified statement of the cost and a half-tone view of the bridge may be found in an article by WILLIAM T. PIERCE in Engineering News, vol. 54, page 265, Sept. 14, 1905.

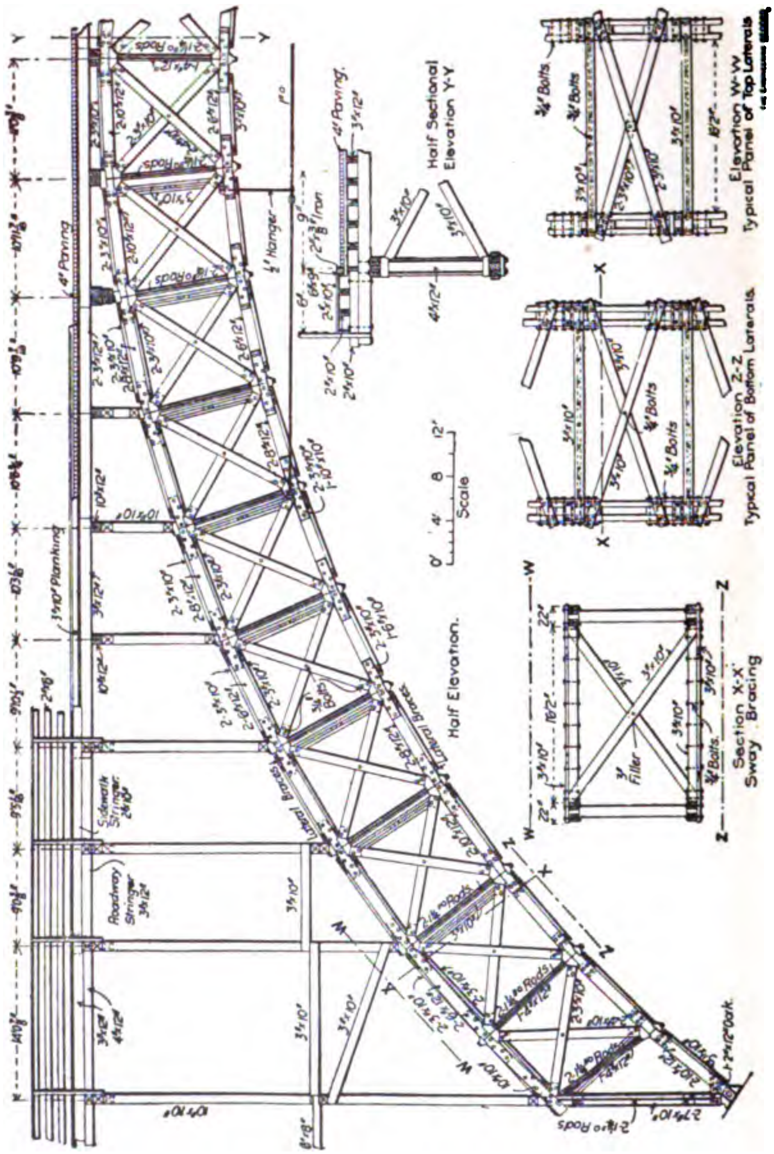


FIG. 74 *f*. Wooden Trussed Arch Bridge over Mendota Ravine, near St. Paul, Minn.

additional information regarding the cost, method of operation, etc., see Engineering News, vol. 54, page 698, Dec. 28, 1905.

Fig. 74*j* gives the half side elevation, two cross-sections, and plans of typical panels of the lateral systems for a highway bridge over the Mendota ravine, near St. Paul. The span of the arch is about 192 feet, and the extreme height about 95 feet. The chords are spliced with plain wooden fish plates at each panel point, and the diagonal struts bear against cast-iron angle blocks. The lower chords are reënforced by horizontal adjustable tension rods connected to them at the fourth panel point from the crown. This illustration is reproduced by permission from Types and Details of Bridge Construction; Part. I, Arch Spans, by FRANK W. SKINNER.

ART. 75. ARCH CENTERING.

Figs. 75*a* to *f* show the general side elevation, central transverse sectional elevation, and certain enlarged details of the centers for the Piney Branch concrete arch bridge in Washington, D.C. The general arrangement of the posts and of some of the bracing differs materially from that of any centers previously constructed in this country, thus creating practically a new type. It is possible to compute with reasonable precision the stresses to which the various posts, struts, and beams are subject. The location of the temporary concrete footings and the varying inclination of the posts were so designed on account of the unique method of erection adopted, and which is described and illustrated in Engineering News, vol. 55, page 453, April 19, 1906. Especial attention is called to the 6 by 6-inch struts between the 12 by 16-inch caps directly below the joists. The transverse bracing of each bent is remarkably simple and effective. The horizontal rails are all double and their ends are connected to the caps by the device illustrated in Fig. 75*f*. The tongued and grooved lagging is arranged by panels, as indicated

(EXAMPLES OF FRAMING. CHAP V.)

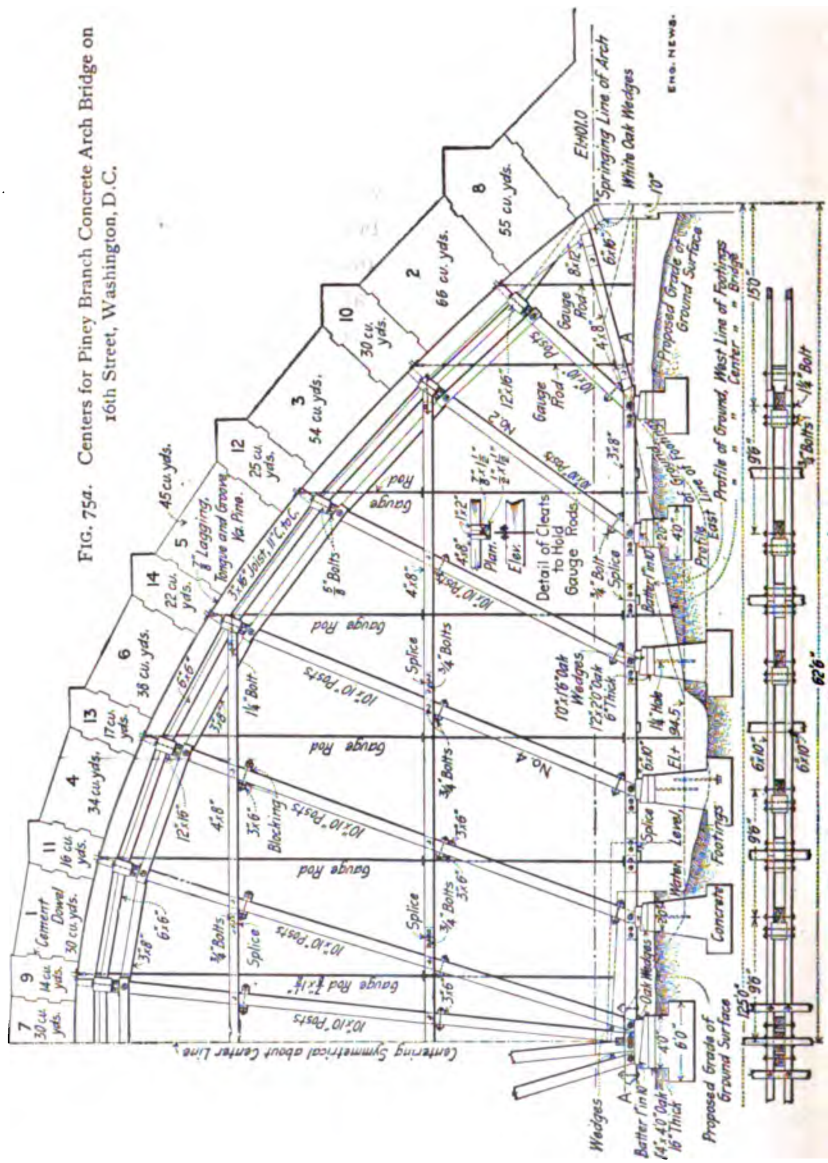
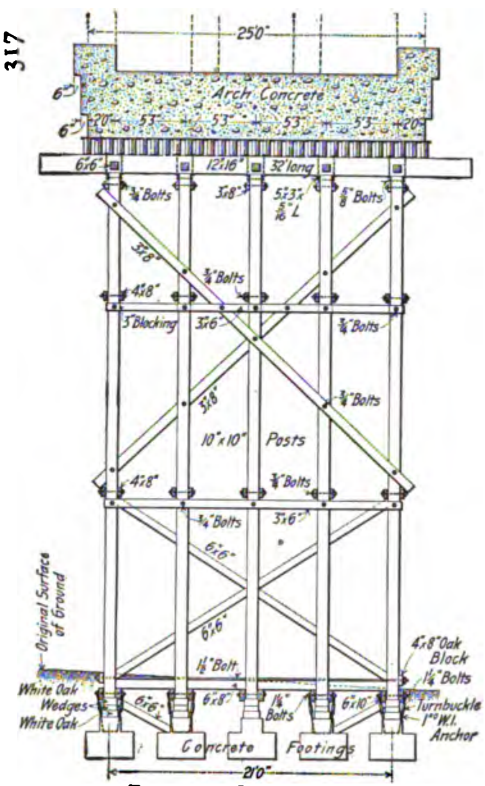
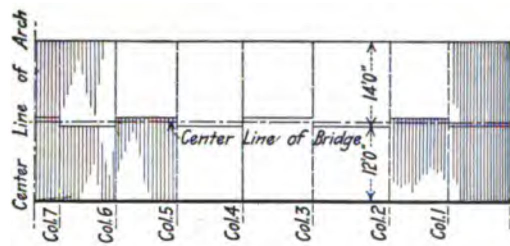


FIG. 75a. Centers for Piney Branch Concrete Arch Bridge on 16th Street, Washington, D.C.

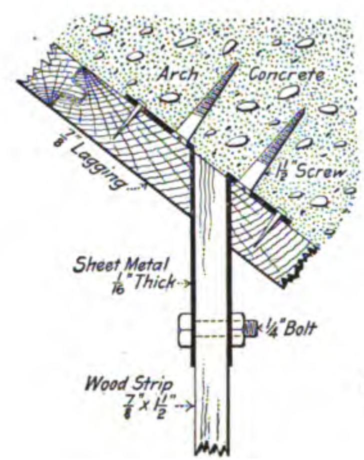


Transverse Section at Center.

FIG. 75*b*.



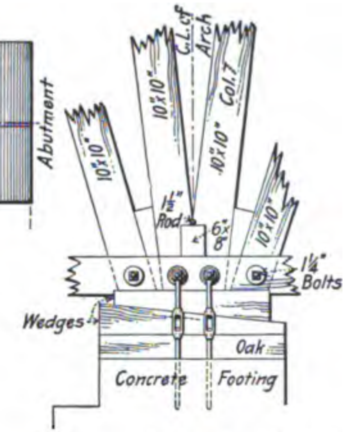
Layout of Lagging; Half Plan.



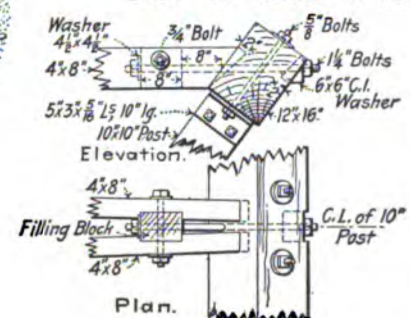
ENG. NEWS.

Detail of Gage Rod.

FIGS. 75*c* and *d*.



Base of Center Posts.



Detail at Cap of Columns 2 and 4.

FIGS. 75*e* and *f*.

in Fig. 75*c*, so as to facilitate the work of demolition. White oak bearing blocks are placed on the footings, and above these the white oak wedges which receive the vertical components of the thrusts of the posts, the horizontal components being resisted by the timbers into which the upper wedges are notched, as shown in Fig. 75*e*.

The gage rods shown in Figs. 75*a* and *d* were used for the purpose of keeping a record of the deformation of the centering during construction, and also of the deformation of the arch ring after construction. The movement of the bottom of each rod, with reference to a fixed elevation near the ground, is read and recorded from time to time so that all changes may be definitely known by the engineer. The deflections of the centers and of the arch ring, as observed by means of the gage rods, are shown graphically in an article published in Engineering News, vol. 57, page 682, June 20, 1907. The results give evidence of the vital importance of the plan adopted (see Fig. 75*b*) of making the distance between the exterior posts of each bent somewhat less than the total width of the bridge, so as to bring practically the same load on each post of a bent whether located at the face or in the middle. Such gage rods were first used by W. J. DOUGLAS during the construction of the Connecticut Avenue concrete arch bridge in the same city.

The centers in Fig. 75*g* were used for a semi-circular stone arch. The main horizontal timber is placed some distance above the springing line of the arch, thereby securing a more satisfactory web system for the upper frame. The lower frame acts somewhat like an arch. Straps and bolts are used as fastenings at most of the joints. Two other designs for centering by W. J. DOUGLAS, and the modified construction as erected by the contractor, are published in Engineering Record, vol. 60, page 172, Aug. 14, 1909.

Fig. 75*h* gives the details of the truss centers used in the con-

struction of the Rockville arch bridge across the Susquehanna river on the Pennsylvania Railroad. Each center consisted of

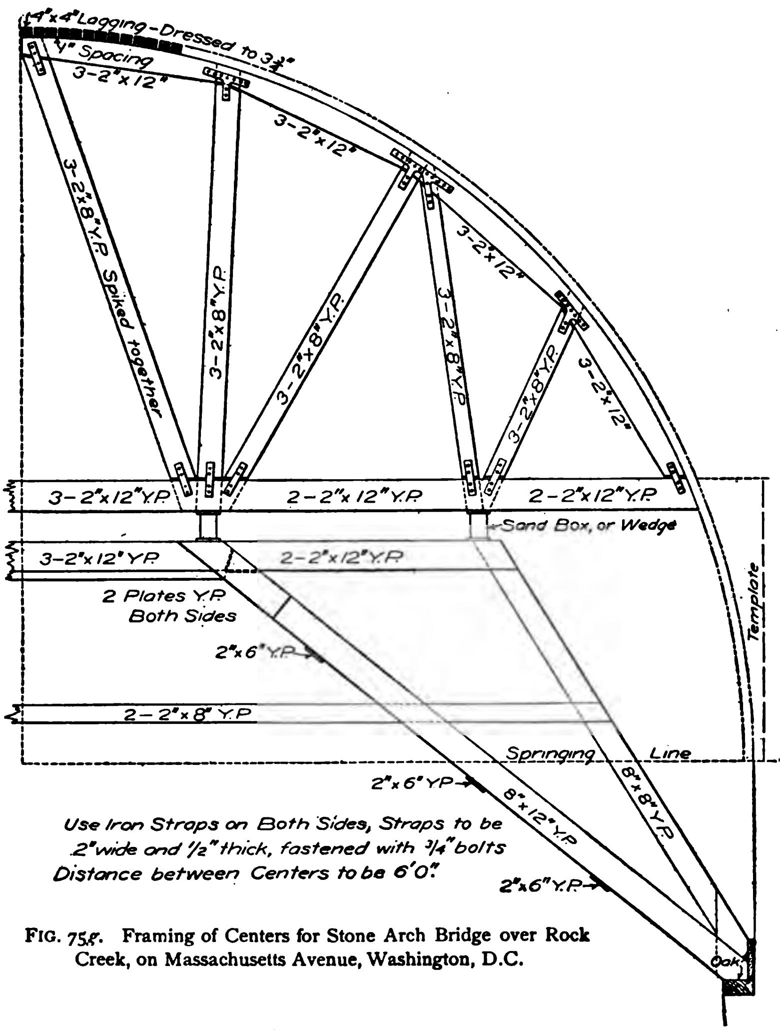


FIG. 75*g*. Framing of Centers for Stone Arch Bridge over Rock Creek, on Massachusetts Avenue, Washington, D.C.

eight parallel trusses with a horizontal lower chord and a segmental upper chord built up of heavy plank. The special features include the cruciform section of the web members, the

steel plate connections at the end joints, and the rollers adjacent to the wedge supports on the two-story pier bents, the sills of the bents resting on the projections of the concrete pier footings. The center had only half the width of the arch ring, and after one-half of the ring was completed the center was lowered

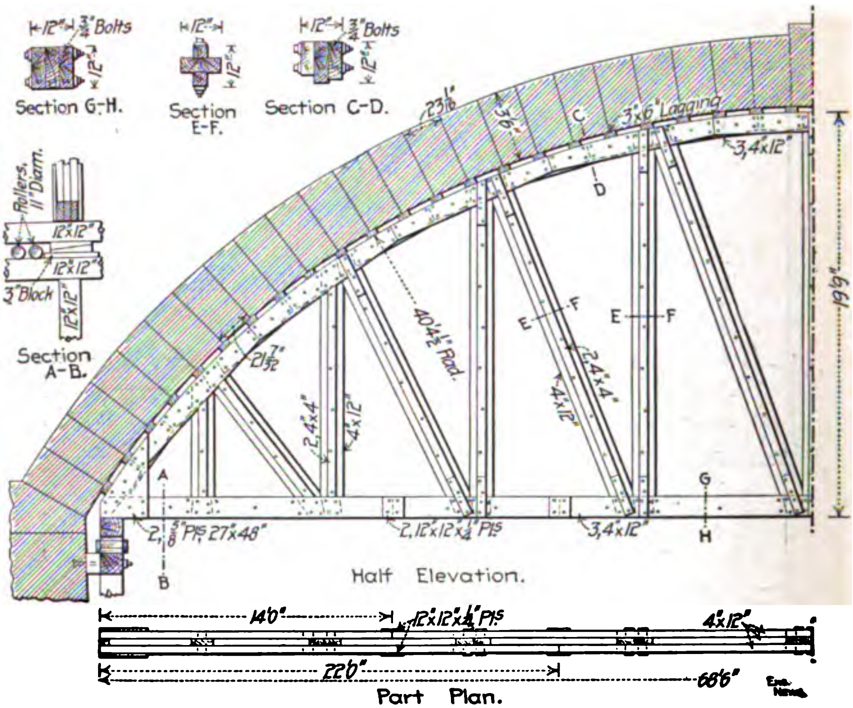


FIG 75*h*. Details of Centers for Rockville Bridge, Pennsylvania Railroad.

on to the rollers by knocking out the wedges and moved over into position for the construction of the second half. The bridge contains 48 spans, each 70 feet long in the clear. Fig. 75*h* is reprinted from Engineering News, vol. 46, page 448, Dec. 12, 1901.

The details of the center shown in Fig. 75*i* are given in Fig. 16*d*. It consists of 8 framed ribs spaced 3 feet 10 inches

apart, center to center, these all being supported on three intermediate bents of 8 piles each, and by blocking on each foundation footing. Lagging strips 4 by 5¾ inches were placed at each voussoir joint. See Engineering News, vol. 49, page 266, March 26, 1903.

An interesting type of trussed center is the one used for the arches in the piers of the Queensboro cantilever bridge over the

FIG. 75*i*. Stone Arch Bridge of C. M. & St. P. Ry., Watertown, Wis.

East River at Blackwell's Island, New York City, the span being 48 feet in the clear. The lower chord or tie beam is 10 by 12 inches, without a splice, the three long radial members consist of two pieces 3 by 10 inches, the two long-chord and the four short-chord members are 10 by 12 inches, the four short braces are composed of two pieces 3 by 10 inches, and the filling piece, above the short upper chord members are 6 by 12, and 6 by 8

FIG. 75*j*. Centers for an Elliptical Arch at 138th Street, Riverside Drive, New York City.

The centering is to support a concrete arch with granite faces, having a span of 50 feet and a rise of 12 feet 6 inches. The centers consist of triangular trusses having the same span, with braces radiating from the upper panel points to the scarf boards on which the lagging is laid. The connections at the panel points are made with bolts. An elevation of the arch reproduced from the architect's drawings and a half cross-section of the arch may be seen in Engineering Record, vol. 51, page 157, Feb. 11, 1905.

inches in section. Single and triple steps are used for the joints. The center rests on temporary stone corbels which are cut away after the arch is completed. See article on Construction of the

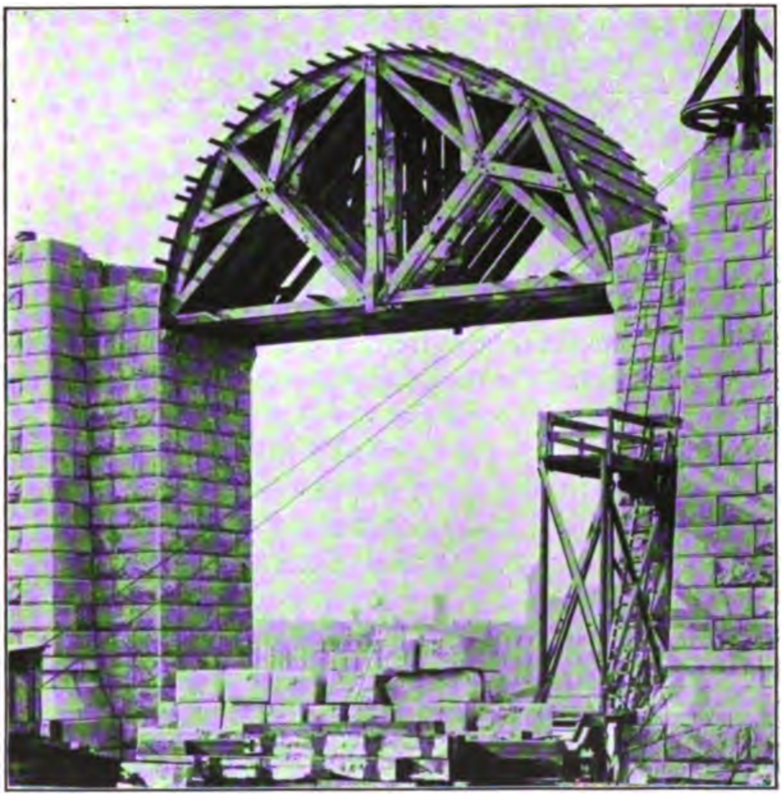


FIG. 75*k*. Center for Pier Arch of Queensboro Bridge, New York.

Blackwell's Island Bridge Masonry, in Engineering Record, vol. 46, page 554, Dec. 13, 1902. Additional references to centering are given in Art. 77.

ART. 76. MISCELLANEOUS STRUCTURES.

The small Howe truss, with a span of 33 feet 4 inches, illustrated in Fig. 76*a*, was used over the main entrance to the

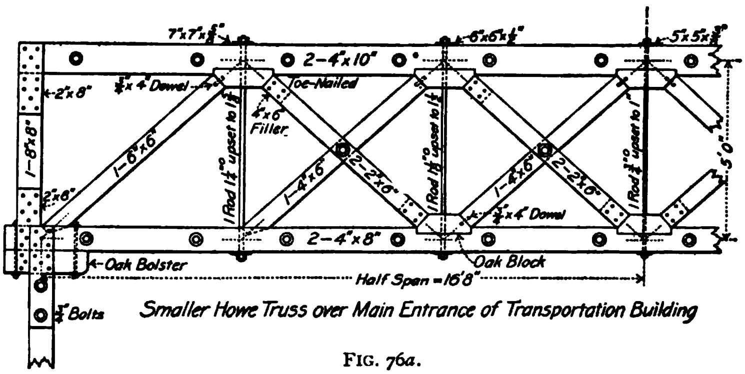


FIG. 76*a*.

Transportation Building of the World's Columbian Exposition at Chicago. The counter diagonals consist of two pieces with filler blocks between them at the ends, and are toe-nailed to the oak angle blocks. The main diagonals are held in position by dowels. Both chords consist of two sticks bolted together at intervals. Plaster joints are used at the end panel points, and at the connection with the supporting posts.

Figs. 76*b* and *c* give the typical details of a flume and supporting trestle on the line of the Dulzura conduit in Southern Cali-

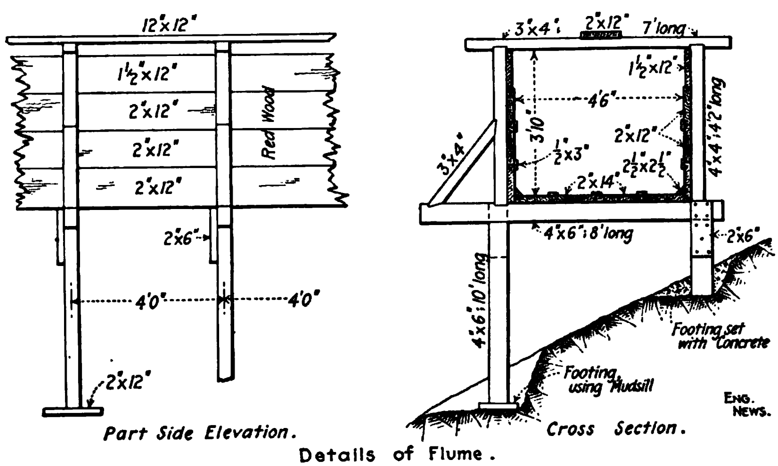


FIG. 76*b*. Flume on Dulzura Conduit, California.

fornia, which is described in an article by M. M. O'SHAUGNESSY, in Engineering News, vol. 60, page 579, Nov. 26, 1908. The trestle work is of pine, on concrete footing blocks, except in a few places where wooden mudsills are indicated in the drawing.

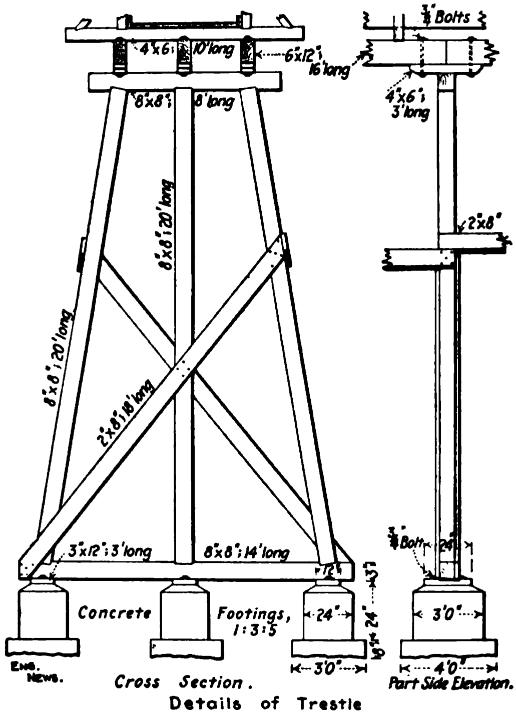


FIG. 76c. Trestle on Dulzura Conduit.

The flume itself is of redwood. Battens cover the butt joints of the 2-inch plank, and triangular strips are put in the corners. The vertical posts of the frames are notched into sills and tie-timbers. The diagonal brace is stepped into the post and sill, and in Fig. 76*b* plaster joints are shown between the sill and its supporting posts.

An interesting example of light framing, in which stiffness

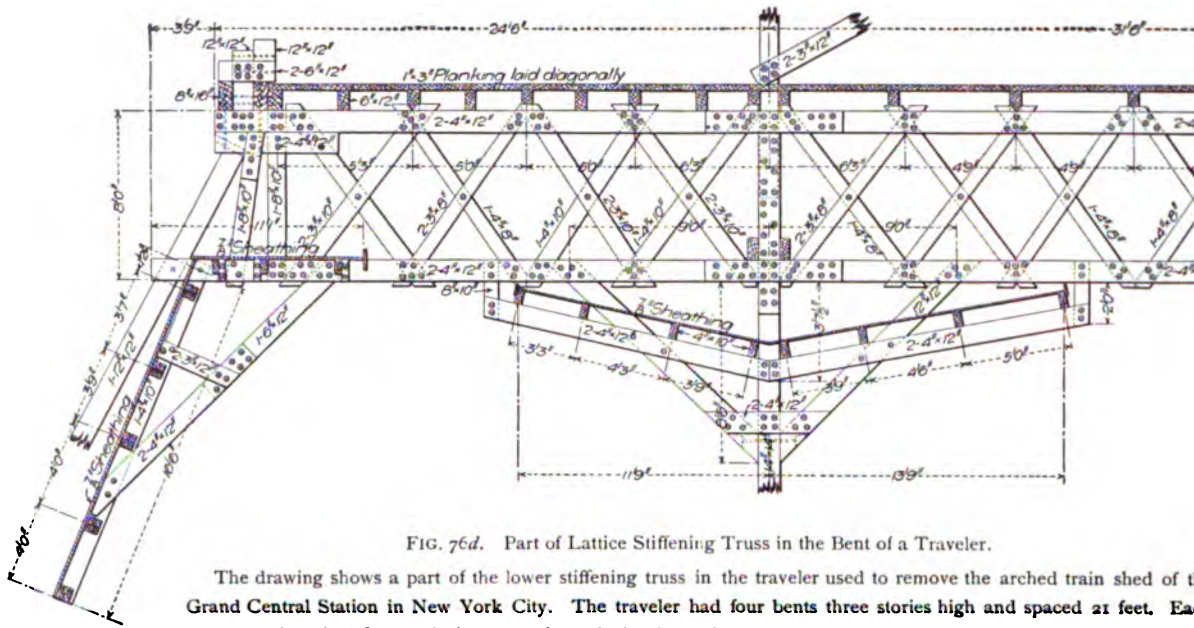


FIG. 76*d*. Part of Lattice Stiffening Truss in the Bent of a Traveler.

The drawing shows a part of the lower stiffening truss in the traveler used to remove the arched train shed of the Grand Central Station in New York City. The traveler had four bents three stories high and spaced 21 feet. Each transverse bent had five vertical posts 14 by 14 inches in section. The connecting lattice trusses had a clearance of 16 feet above the track. The truss supports a working platform built strong enough to carry heavy concentrated or uniform loads. It is connected to each post by long knee braces, and supports sheathing over the platforms between the tracks. The end diagonal is a 12 by 12-inch timber extended to supports on the ground, forming inclined shores which are wedged up at the lower end when the traveler is in service. See Engineering Record, vol. 58, page 321, Sept. 19, 1908.

rather than strength is the most important consideration, is that of the temporary signals built by the Coast and Geodetic Survey of the United States, as described and illustrated in Appendix No. 4 to the Report of 1903, on Triangulation Southward along the Ninety-eighth Meridian in 1902, by JOHN F. HAYFORD.

Oblique scarf joints are used for splicing the long posts of both tripod and scaffold which compose the signal, the length of the splice being about 6 times the thickness of the timber. For a height of 60 feet, 5 by 5-inch scantling is used for the posts and 2 by 4-inch scantling for the horizontal and diagonal braces. The end of each stick is $\frac{3}{4}$ inch thick after the cut is made for the splice. The scarf joints are nailed with 60-penny nails, each brace whether horizontal or diagonal is fastened to each post by two 40-penny nails, and one 40-penny nail is used at each intersection of the diagonals. The extra stiffness required is secured by springing the posts before the braces are nailed into final position, thus subjecting the posts to initial flexure, and the horizontal braces to initial tension or compression. Additional details regarding the proportions of the tripod and scaffold and a description of the methods of erection and anchorage may be obtained by consulting the reference given above.

Fig. 76*e* shows a cross-section of the combined terminal freight house at Galveston, Tex., of the Rock Island and the Colorado & Southern Railways. The freight house is 46 by 300 feet, with a platform at its west end 46 by 200 feet. The building has a second story for offices for 110 feet of its length, the end elevation of which is also given in the illustration. It is built of brick, with composition roofing, and with concrete footings on a pile foundation. The inclined 10 by 14-inch yellow pine transverse beams support 6 by 8-inch purlins on which the sheeting is laid. The beams are connected to the supporting

post by a keyed bolster and by angle braces having single step joints. See Railroad Age Gazette, vol. 45, page 996, Sept. 25, 1908.

The falsework used in 1909 to erect the viaduct approaches of the Manhattan Bridge at New York City consists of wooden vertical posts and horizontal struts combined with adjustable steel diagonals. The connecting details at the joint are com-

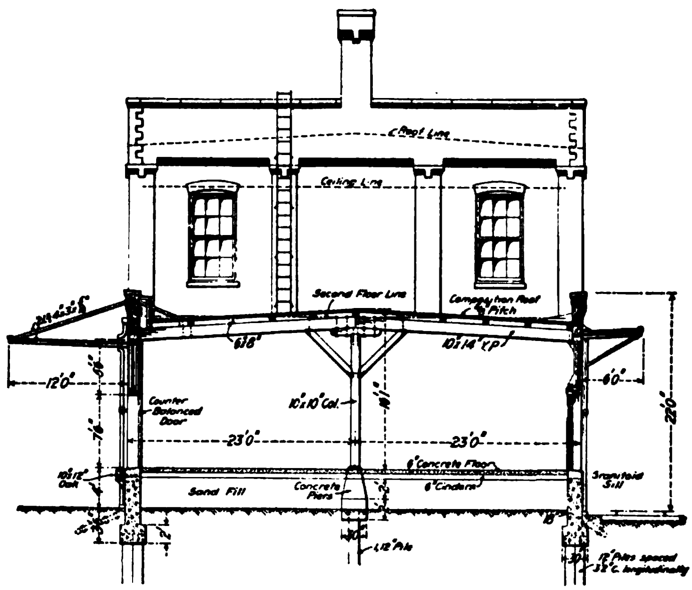


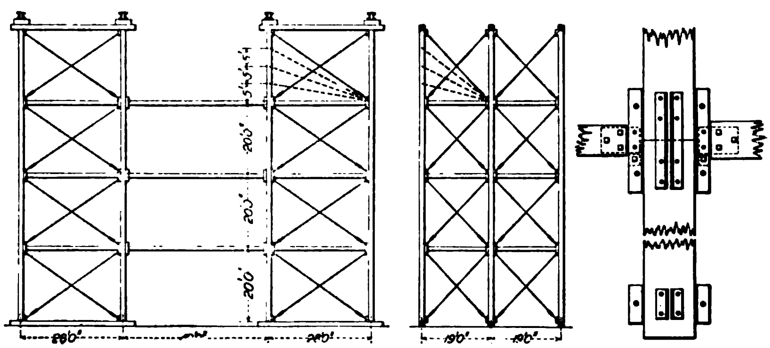
FIG. 76e. Cross-section through Freight House.

posed of steel angles, plates, pins, and bolts. As indicated in Fig. 76*f* the lower three stories are of fixed height and an upper story of variable height, corresponding to the required elevation, and changing with the grade.

The connecting details are shown in the right-hand diagram, or Fig. 76*g*. The vertical posts consist of timber 14 inches square, have butt joints, and are spliced on each face by pairs of 3 by 2-inch vertical angles about 27 inches long, secured by four ¾-inch bolts through each angle. The angles of each pair are

spaced 1 inch apart in the clear, to receive the vertical ½-inch plates attached to the ends of its horizontal struts. The outstanding legs of the angles have holes for the ¾-inch bolts to connect them to these plates and angles, and also holes to receive the 1¼-inch pins, which pass through the loop eyes of the ⅞-inch diagonal rods. The vertical plates are connected to the struts by insertion in slots at the ends of the timbers and fastened with three ¾-inch bolts.

The system of connections is simple and efficient, facilitates rapid assembling and dismantling without injury to the material,



FIGS. 76*f* and *g*. Combination Falsework for erecting Viaduct Approaches.

thus permitting it to be used over and over again, and secures considerable economy in handling as well as in the ultimate cost of the material, including freight charges. The connections are made symmetrical so that the corresponding members are interchangeable in the lower stories of fixed height. In case of uneven settlement under the posts the diagonals are adjusted so as to relieve undue stress. Accordingly the members can be accurately proportioned for the stresses required to support the known loads. The preceding facts and illustration are taken from an article on Progress on the Manhattan Bridge, in Engineering Record, vol. 60, page 116, July 31, 1909. See also an editorial on Combination Falsework on page 113.

The queen-post truss in Fig. 76*h* was designed by RALPH MODJESKI to be used at each end of a pair of barges to hold them together during the construction of the pneumatic caissons for the Willamette river bridge at Portland, Ore., of the Northern Pacific Railway. Fig. 76*i* gives a view taken Sept. 8, 1906, of the caisson during construction, supported on beams which rest on the barges, and of the braced frames from which it is lowered to a floating position. It will be observed that the vertical members consist of two tie rods each, and which pass outside of the upper chord and take bearing on overhanging bent washers.

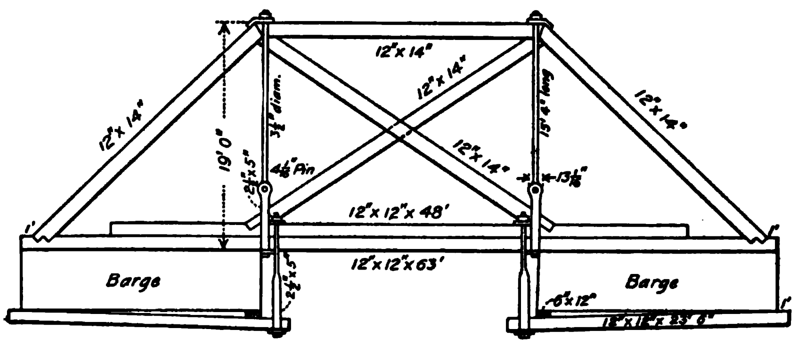


FIG. 76*h*. Queen-post Truss.

On the sections of the Rapid Transit Subway extending from 60th to 104th Streets on Broadway, New York City, triangular wooden trusses with four panels were used to support the tracks for the surface cars during the excavation and construction for the subway. The view in Fig. 76*j* was taken at the corner of Broadway and 92d Street. Transverse timbers spaced 5 feet apart, extending under the conduit and track, were supported by bolt hangers from the lower chords of the trusses, having a span of nearly 60 feet. The panels next to the middle are 12½ feet long. The upper chords and diagonal struts are 12 by 12-inch timbers, and the lower chords are either 16 by 16 or 20 by 20 inches in section. The middle vertical consists of a 12 by 12-

FIG. 76*i*. Barges supporting Pneumatic Caisson under Construction at Portland, Ore.

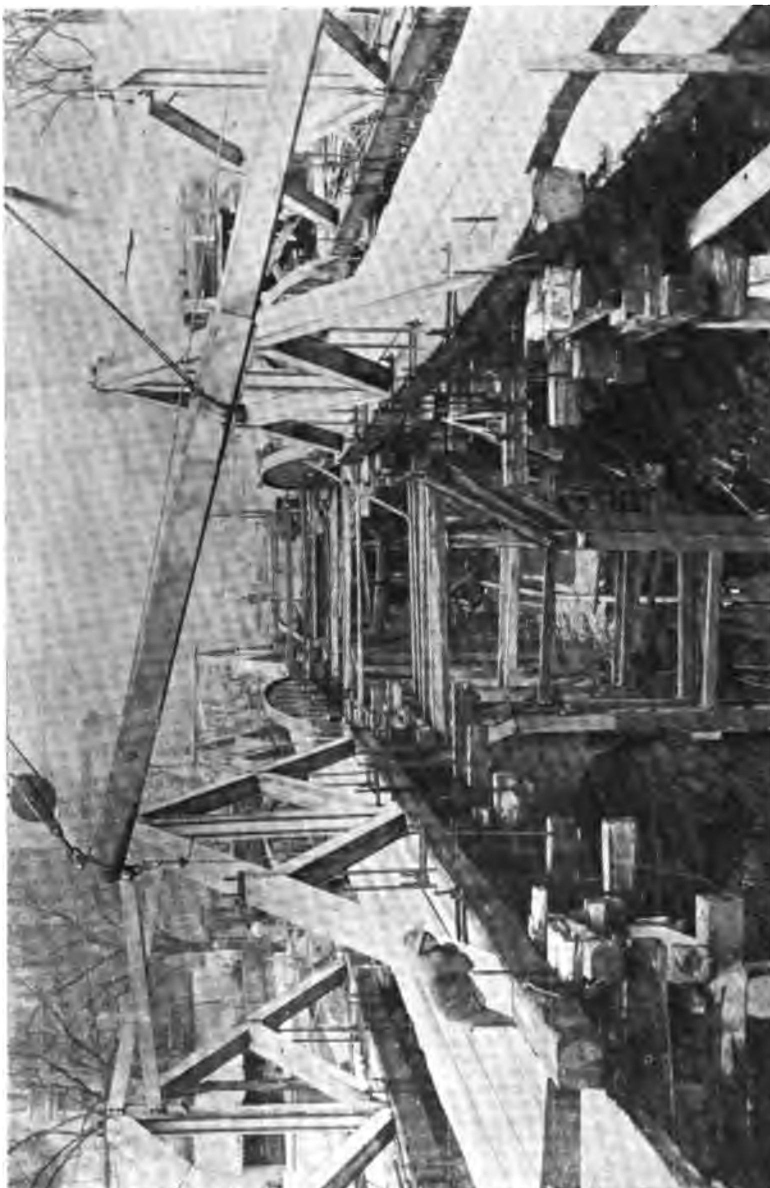


FIG. 76j. Trusses supporting Street Railway Tracks during Construction of New York Subway.

inch timber with two pairs of $1\frac{3}{4}$-inch round rods on each side, its upper end being connected to the upper chord timbers by two steel plates and bolts. The bearing plate on top is 12 by $\frac{3}{4}$ inches and 17 inches long. The side verticals consist of two $1\frac{3}{4}$-inch round rods each. The lower chord is subject to combined tension and flexure. The trusses are spaced 11 feet apart in the clear, and braced together in the middle high enough to clear the street cars. The manner in which the trusses were

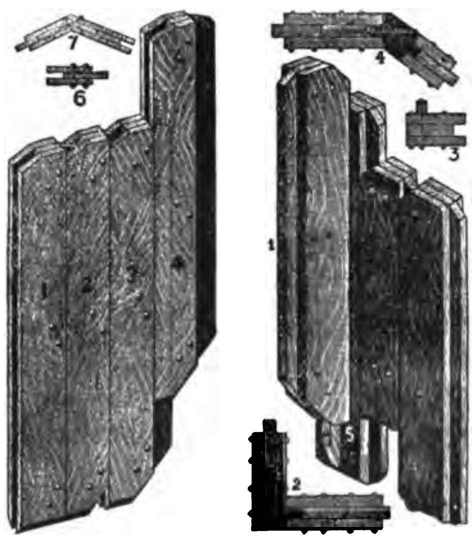


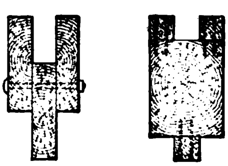
FIG. 76*k*. Wakefield Sheet Piling.

successively supported during the excavation and subsequent construction is described in articles on Sections 6A and 6B, New York Rapid Transit Railway, in Engineering Record, vol. 43, page 294, March 30, 1901; and Engineering News, vol. 48, page 390, Nov. 13, 1902.

An example of simple framing which has been very successfully employed since 1889 in the construction of sheet pile

cofferdams, consists essentially of three planks of equal width bolted and spiked together so as to form a tongue and groove (Fig. 76*k*). The center plank is sized to a regular thickness to insure a good fit between the tongues and grooves of adjacent piles. A $2\frac{1}{2}$-inch tongue is used for 1-inch boards, and a 4-inch tongue on 4-inch planks. The corresponding sizes of bolts are $\frac{3}{8}$ and $\frac{5}{8}$ inch in diameter. Two bolts are staggered in every 5 to 8 feet of the length of the pile, spikes being used between the bolts on long piles.

These compound sheet piles have a much greater resistance to impact than a pile of the same gross cross-section composed of a single stick. The method of sharpening the pile in order to keep it in line, and to keep close to the adjacent one during driving, is shown in the illustration at the left. A view of a rectangular corner is shown at 1, and the sectional plan at 2, a tongue being bolted to the side of a pile when the corner is reached, as at 3 and 5. Any other angle is turned as indicated at 4 or 7. The relatively long tongue and groove is indicated at 6. Many examples of its use are described in Ordinary Foundations, by CHARLES EVAN FOWLER. See editorial in Engineering News, vol. 23, page 199, March 1, 1890. Fig. 76*m* shows a section of a large pile in which the tongue and groove are formed by spiking separate pieces to the large timber.



FIGS. 76*l* and *m*. Sections of Sheet Piles.

ART. 77. REFERENCES TO ENGINEERING LITERATURE.

The following selected references relate to the subjects of the preceding articles in this chapter. As indicated in Art. 71, their purpose is merely to illustrate methods and details of framing, and to serve as an introduction for further study by the student. It will be a valuable exercise to supplement these lists by references to other periodicals arranged in card catalogue form.

SLOW-BURNING CONSTRUCTION.

Mills and Mill Engineering, by Edward Sawyer, with discussion by John R. Freeman and others. — Eng. Rec., v. 21, p. 148, March 22, 1890.

Standard Storehouse Construction, by C. J. H. Woodbury. — Eng. Rec., v. 24, p. 246, Sept. 19, 1891.

Lessons of the Park Place Disaster, by Edward Atkinson. — Eng. Mag., v. 2, p. 137, November, 1891.

Some Notes on Fire Protection Engineering, by John R. Freeman. — Trans. Assoc., C. E. Cornell, 1894, v. 2, p. 85.

Our Enormous Annual Loss by Fire; A Waste Born of Ignorance and Neglect, by Edward Atkinson. — Eng. Mag., v. 7, p. 603, August, 1894.

Steel Beams in a Textile Mill. — Eng. Rec., v. 33, p. 316, April 4, 1896.

Construction of Slow-burning Buildings, by Francis C. Moore. — Eng. Mag., v. 14, p. 955, March, 1898.

Designs of Factory Buildings, by Hales & Ballinger. — Eng. Rec., v. 44, p. 505, Nov. 23, 1901.

Comparative Cost of Wood and Steel Frame Factory Buildings, by H. G. Tyrrell. — R. R. Gaz., v. 37, p. 437, Oct. 4, 1904.

Improved Method of Saw-tooth Roof Construction, by Samuel M. Green. — Eng. News, v. 60, p. 263, Sept. 3, 1908; Eng. Rec., v. 55, p. 53, Jan. 12, 1907.

TRESTLE CONSTRUCTION.

Design of Wooden Trestles. — Editorial in Eng. News, v. 20, p. 50, July 21, 1888.

Use of Wood in Railroad Structures, Chap. XIX. Details of Trestle Construction, by Chas. D. Jameson. — R. R. & Eng. Jour., v. 4, p. 21, January, 1890.

Construction of the St. Louis, Peoria & Northern Railway, by F. G. Jonah. — Eng. News, v. 43, p. 110, Feb. 15, 1890.

High Trestles of the Esquimalt & Nanaimo Railway, by Jos. Hunter. — R. R. Gaz., v. 23, p. 89, Feb. 6, 1891.

Temporary Trestle, by Robert A. Cummings. — R. R. Gaz., v. 26, p. 467, June 29, 1894.

Rocky Mountain Work on the Great Northern, by Jas. H. Kennedy. — R. R. Gaz., v. 26, p. 696, Oct. 12, 1894.

Trestle for Single Track Electric Railway. — Eng. News, v. 35, p. 255 (inset), April 16, 1896.

American Railway Bridges and Buildings, by Walter G. Berg. 1898, pp. 9–11, 198, 206, 207, and 436.

Thomson Run Bridge. — R. R. Gaz., v. 31, p. 689, Oct. 6, 1899.

Reconstruction of the Utah Central Railway, by W. P. Hardesty. — Eng. News, v. 45, p. 44, Jan. 17, 1901.

Railway Trestle with Bents of Reinforced Concrete, by Wm. A. Allen. — Eng. News, v. 49, p. 244, March 12, 1903.

Construction of the Texas & Oklahoma Railroad, by Robert F. Powell. — Eng. Rec., v. 49, p. 296, March 5, 1904.

General Plans of Framed Trestles on Intramural Railway. — Eng. Rec., v. 49, p. 578, May 7, 1904.

High Timber Trestle on the North Alabama Railroad. — Eng. Rec., v. 50, p. 110, July 23, 1904.

Building of the Chicago, Cincinnati & Louisville, and Cincinnati, Richmond & Muncie Railroads, by H. L. Weber and Fred R. Charles. — Eng. Rec., v. 51, p. 64, Jan. 21, 1905.

Standard Plans for Pile and Timber Trestle Bridges; Santa Fe Railway System, by A. F. Robinson. — Proc. Am. Ry. Eng. & M. of W. Assoc., 1905, v. 6, p. 43.

Wooden Trestle Bridges with Ballast Floors. — Proc. Am. Ry. Eng. & M. of W. Assoc., 1908, v. 9, p. 320; 1906, v. 7, p. 705.

Standard Trestle Bridge; Denver, Northwestern & Pacific. — R. R. Gaz., v. 40, p. 503, May 18, 1906.

Hints on the Design and Construction of Wooden Trestles, by R. Balfour. — Eng. News, v. 57, p. 610, June 6, 1907.

Fort Dodge, Des Moines & Southern Railway. — R. R. Gaz., v. 43, p. 680, Dec. 6, 1907.

Design and Construction of Automatic Car Systems, by Charles J. Steffens. — Eng. News, v. 59, p. 165, Feb. 13, 1908.

Large Wooden Trestle at McGill, Nevada, by L. J. Dobbins. — Eng. News, v. 59, p. 409, April 16, 1908.

Large Timber Trestles on the Pacific Coast Extension of the Chicago, Milwaukee & St. Paul Railway. — Eng. News, v. 61, p. 307, March 25, 1909.

Protection of Members on Wooden Bridge Structures. — Eng. Rec., v. 59, p. 637, May 15, 1909.

SMALL BRIDGE TRUSSES.

Queen-post Truss. Trestle for Single-track Electric Railway. — Eng. News, v. 35, p. 255 (inset), April 16, 1896.

Erection of the Park Avenue Viaduct, New York; Part 1. — Falsework Supporting Temporary Tracks and New Center Girders. — Eng. Rec., v. 35, p. 269, Feb. 27, 1897.

Ornamental Bridges in a Private Park. — Eng. Rec., v. 47, p. 556, May 23, 1903.

Trestle Approaches and Skew Spans in Bridge Ten near Liberal Arts Building. — Eng. Rec., v. 49, p. 579, May 7, 1904.

Design of Timber Howe Trusses, by R. Balfour. — Eng. News, v. 58, p. 224, Aug. 29, 1907.

Weehawken Transfer Bridges of the West Shore Railroad. — Eng. Rec., v. 57, p. 185, Feb. 15, 1908.

Pontoon or Floating Drawbridges. — Eng. News, v. 59, p. 474, April 30, 1908.

Highway Bridge across the Kansas River at Fort Riley, Kansas, by P. S. Pond. — Eng. Rec., v. 58, pp. 44, 75, July 11, 18, 1908.

[Standard Design for Typical Wooden Overhead Highway Bridge, Single Track Crossing], Carolina, Clinchfield & Ohio

Railway, by George L. Fowler.—R. R. Age Gaz., v. 46, p. 539, March 19, 1909.

ARCH CENTERING.

Arch Centers, by James W. Rollins, Jr.—Jour. Assoc. Eng. Soc., v. 27, p. 9, July, 1901.

Melan Arch Park Bridges at Washington, D.C., by W. J. Douglas.—Eng. News, v. 46, p. 323 (inset), Oct. 31, 1901.

Boulder-faced Melan Arch Bridge, National Park, Washington, D.C., by W. J. Douglas.—Eng. News, v. 48, p. 109, Aug. 14, 1902.

Concrete-steel Bridge at Plano, Ill., Chicago, Burlington & Quincy Railway.—Eng. News, v. 52, p. 559, Dec. 22, 1904; Eng. Rec., v. 49, p. 18, Jan. 2, 1904; R. R. Gaz., v. 35, p. 886, Dec. 11, 1903.

Stone Arch Bridge on the Chicago, Milwaukee & St. Paul Railway at Watertown, Wis.—Eng. News, v. 49, p. 266, March 26, 1903 (see Figs. 16*d* and 75*i*).

Concrete Arch Bridge with Bar and Stirrup Reinforcement.—Eng. News, v. 50, p. 588, Dec. 31, 1903.

Improvements on the Big Four Line between Lawrenceburg Junction and Sunman —Eng. Rec., v. 49, p. 292, March 5, 1904.

Permanent Way and Structures of the San Pedro, Los Angeles & Salt Lake.—R. R. Gaz., v. 37, p. 240, Aug. 12, 1904.

Structural Details of the New Reinforced Concrete Bridge at Grand Rapids, Mich., by Wm. F. Tubesing.—Eng. News, v. 52, p. 489, Dec. 1, 1904.

Reinforced Concrete Arch Bridges, Como Park, St. Paul.—Eng. Rec., v. 50, p. 648, Dec. 3, 1904.

Reinforced Concrete Foot Bridge at Como Park, St. Paul, Minn.—Eng. News, v. 53, p. 352, April 6, 1905.

Connecticut Avenue Concrete Arch Bridge, Washington, D.C.—Eng. Rec., v. 52, p. 30, July 8, 1905.

Concrete Viaducts of the Thebes Bridge. — Eng. Rec., v. 52, p. 128, July 29, 1905.

Concrete Viaduct at Riverside, Cal., by H. Hawgood.— Eng. Rec., v. 52, p. 284, Sept, 9, 1905.

Reinforced Concrete Double-track Railroad Arch Bridge.— Eng. Rec., v. 52, p. 295, Sept. 9, 1905.

Pollasky Reinforced Concrete Bridge. — Eng. Rec., v. 53, p. 226, Feb. 24, 1906.

Danville Arch Bridge of the Cleveland, Cincinnati, Chicago & St. Louis Railway. — Eng. Rec., v. 53, p. 238, March 3, 1906.

Arch Rib Bridge of Reinforced Concrete at Grand Rapids, Mich., by George Jacob Davis. — Eng. News, v. 55, p. 321, March 22, 1906.

Reinforced Concrete Arch Bridge at Peru, Ind., by Daniel B. Luten. — Eng. News, v. 55, p. 347, March 29, 1906.

Concrete Arch on the Big Four at Danville. — R. R. Gaz., v. 41, p. 30, July 13, 1906.

Special Form of Arch Centering, by J. H. Milburn. — Eng. News, v. 56, p. 207, Aug. 23, 1906.

Construction of the 175th Street Arch, New York City. — Eng. Rec., v. 56, p. 379, Oct. 5, 1907.

Design of the Centering for the 233-Foot Arch Span, Walnut Lane Bridge, Philadelphia, Pa., by George Maurice Heller. — Proc. Engrs. Club of Philadelphia, v. 25, p. 257, July, 1908.

Wyoming Avenue Arch Bridge at Philadelphia. — Eng. Rec., v. 59, p. 233, Feb. 27, 1909.

Construction of the Edmondson Avenue Bridge, Baltimore. — Eng. Rec., v. 60, p. 172, Aug. 14, 1909.

MISCELLANEOUS STRUCTURES.

Burnett & Clifton Automatic Coal Chute for Coaling Locomotives. — Eng. News, v. 26, p. 604 (inset), Dec. 26, 1891.

Memphis Bridge by George S. Morison, 1894. — Plates 11, 12, 13, 14, 16, 17, 19, 20, 21.

Wooden Water Tower [for a Country Estate]. — Eng. Rec., v. 37, p. 473, April 30, 1898.

Falsework for Erecting the Manhattan Towers and End Spans; New East River Bridge, by C. E. Fowler. — Eng. News, v. 43, p. 164 (inset), March 8, 1900.

Erection of the Northwest Miramichi Bridge, Newcastle, N.B., by H. D. Bush. — R. R. Gaz., v. 35, p. 58, Jan. 23, 1903.

Baltimore & Ohio Export Pier at Locust Point. — R. R. Gaz., v. 36, p. 196, March 11, 1904.

Building of a World's Fair, How the Louisiana Purchase Exposition has been Constructed. — Eng. Rec., v. 49, p. 568, May 7, 1904.

Building Construction of the St. Louis Exhibition. — Eng. News, v. 51, p. 478, May 19, 1904.

Erection of the 232-Foot Triangular Span of the Fraser River Bridge. — Eng. Rec., v. 56, p. 192, Aug. 13, 1904.

Baltimore Terminal of the Western Maryland R. R. — Eng. Rec., v. 52, p. 4, July 1, 1905.

New St. Louis Freight Terminals of the Wabash. — R. R. Gaz., v. 39, p. 604, Dec. 29, 1905.

Steamship Terminal with Concrete Pile Piers at Brunswick, Ga.; Atlantic & Birmingham Railway. — Eng. News, v. 56, p. 654, Dec. 20, 1906.

Floating Derrick for Handling Heavy Rip-rap Stone. — Eng. News, v. 57, p. 560, May 23, 1907.

Problem in Underpinning. — Eng. Rec., v. 56, p. 94, July 27, 1907.

[Details of 32-Foot Span Timber Flume.] Kern River No. 1 Power Plant of the Edison Electric Co., Los Angeles, by C. W. Whitney. — Eng. Rec., v. 56, p. 140, Aug. 10, 1907.

Roundhouse of the Lehigh and Hudson River Railway at Warwick, N.Y. — Eng. Rec., v. 57, p. 150, Feb. 8, 1908.

[Suspended Needlebeams for Underpinning the Schermerhorn

Building.] Development of Building Foundations, by Frank W. Skinner. — Eng. Rec., v. 57, p. 412, April 4, 1908.

Closure of the Charles River Dam, by Edward C. Sherman. — Eng. News., v. 60, p. 498, Nov. 5, 1908.

Municipal Ferry House Substructure, New York. — Eng. Rec., v. 58, p. 525, Nov. 7, 1908.

Traveler with Pin-connected Braces, Mercantile Bridge across the Monongahela River. — Eng. Rec., v. 58, p. 686, Dec. 19, 1908.

Some Problems in Designing the Kern River No. 1 Hydro-electric Power Plant, by F. C. Finkle. — Eng. News, v. 60, p. 701, Dec. 24, 1908.

[Caisson, Cofferdam, and Masonry, for Pier No. 26, of the New Clinton Bridge.] New Bridge crossing the Mississippi River at Clinton, Ia.; Chicago & Northwestern Railway, by F. H. Bainbridge. — Eng. News, v. 61, p. 63, Jan. 21, 1909.

Gravel Washing Plant on the Richmond, Fredericksburg & Potomac R. R., by E. M. Hastings. — Eng. News, v. 61, p. 406, April 15, 1909.

CHAPTER VI.

TIMBER TESTS AND UNIT-STRESSES.

ART. 78. COMMERCIAL SIZES AND GRADES.

In order to simplify the designs described in preceding chapters for the use of students it has been assumed in the examples given that the timbers were cut to full size. This is not the case under ordinary commercial conditions. It is therefore necessary either to use the actual commercial sizes in making the computations for any design or to modify the allowable working unit-stresses so as to secure practically the same result. The designer must study the conditions of the lumber market in the region where he is located.

As an illustration of the classification of lumber the following quotation is made from the interstate rules adopted in 1905 for yellow pine by the Georgia-Florida Sawmill Association, and various lumber associations, exchanges, etc.

"Flooring shall embrace four, five and six quarter inches in thickness by three to six inches in width, excluding $1\frac{1}{2} \times 6$. For example, 1×3, 4, 5, and 6; $1\frac{1}{4} \times 3$, 4, 5, and 6; $1\frac{1}{2} \times 3$, 4, and 5.

"Boards shall embrace all thicknesses under one and a half inches by over six inches wide. For example: $\frac{3}{4}$, 1, $1\frac{1}{4}$, and $1\frac{3}{8}$ inches thick by over 6 inches wide.

"Plank shall embrace all sizes from one and one-half to under six inches in thickness by six inches and over in width. For example: $1\frac{1}{2}$, 2, $2\frac{1}{2}$, 3, $3\frac{1}{2}$, 4, $4\frac{1}{2}$, 5, $5\frac{1}{4}$, $5\frac{3}{4} \times 6$ and over in width.

"Scantling shall embrace all sizes exceeding one and one-half inches and under six inches in thickness, and from two to under six inches in width. For example: 2 × 2, 2 × 3, 2 × 4, 2 × 5, 3 × 3, 3 × 4, 3 × 5, 4 × 4, 4 × 5, and 5 × 5.

"Dimension sizes shall embrace all sizes six inches and up in thickness by six inches and up in width. For example, 6 × 6, 6 × 7, 7 × 7, 7 × 8, 8 × 9, and up.

"Stepping shall embrace one to two and a half inches in thickness by seven inches and up in width. For example: 1, 1¼, 1½, 2, and 2½ × 7, and up in width.

"Rough Edge or Flitch shall embrace all sizes one inch and up in thickness by eight inches and up in width, sawed on two sides only. For example: 1, 1½, 2, 3, 4, and up thick by 8 inches and up wide, sawed on two sides only."

The standard sizes for dimension lumber adopted by many lumber associations have both the smaller and the larger dimension any inch from 6 inches up. Since the large mills carry practically no stock but always saw to order, any of these section dimensions may be secured, but it is customary for dealers to order from the mills for the purpose of keeping in stock those sizes in which both dimensions are even inches. If a design is made involving a large amount of lumber, it is accordingly economical to order the exact sizes desired for the construction.

Rough timbers sawed to standard size means that they shall not be over ¼ inch scant from the actual size specified. In the smaller pieces, like joists, for instance, this means a large percentage of reduction in strength from that of full size; a joist called 2 × 6 measures 1¾ × 5¾ inches and has closely 80 percent of the strength of a stick measuring 2 × 6 inches, and 77 percent of its stiffness.

Standard dressing means that not more than ¼ inch shall be allowed for dressing each surface. For instance, a 12 × 12-inch

timber dressed on four sides shall not measure less than $11\frac{1}{2} \times 11\frac{1}{2}$ inches. For the small sizes this reduction in size involves a large reduction in strength and stiffness. For example, according to the standard rules for yellow pine lumber a 2×6 S1S1E (surfaced on 1 side and 1 edge) is worked to $1\frac{5}{8} \times 5\frac{5}{8}$ inches, a 2×8 S1S1E to $1\frac{5}{8} \times 7\frac{1}{2}$ inches. The former has a strength and stiffness of about 71 and 67 percent respectively of a joist measuring exactly 2×6 inches. For S4S (surfaced on 4 sides) the dimensions are $\frac{1}{8}$ inch less than those given for S1S1E.

The standard lengths in the United States are multiples of 2 feet. In many cases standard lengths do not exceed 20 feet, although greater lengths may be secured on special order. In Canada the standard lengths are multiples of 1 foot.

The rules regarding classification of sizes differ in various parts of the country, but the tendency is toward greater uniformity through the coöperation of lumber manufacturers' associations.

The rules regarding the quality of lumber, the definitions of defects, and to what extent defects are allowed, are known as grading rules. Different grades are known by selected trade names such as standard heart, square edge, etc. An historical account of the development of grading in this country, together with copies of the grading rules, were published in 1906 in Bulletin 71 of the U. S. Forest Service, entitled Rules and Specifications for the Grading of Lumber Adopted by the Various Lumber Manufacturing Associations of the United States, compiled by E. R. HODSON. These rules are subject to revision as natural and commercial conditions may require. On the inspection of timber the student is referred to BYRNE's Inspector's Pocket-Book.

As examples of the relation of differences in cost to those in size the following general statements relating to railroad and shop material in longleaf yellow pine are given. For lengths up

to 20 feet, timbers 10 × 12 and 12 × 12 inches may cost from 20 to 30 percent more than 2 × 4 to 8 × 8 inches, depending upon the grade. For widths over 12 inches up to 14 inches add \$2.00 per M feet; for 14 to 16 inches, \$4.50; for 16 to 18 inches, \$7.00. For each 2 inches in thickness over 12 inches, add \$1.00. For lengths of 24 feet add \$1.00 to the price per M feet for a length of 20 feet, and for each additional length of 2 feet add \$1.00, up to a length of 36 feet. The size of stringers for trestles in most general use are 6, 7, and 8 inches in width, and 14, 15, and 16 inches in depth. The prices for depths of 15 and 16 inches are respectively about \$1.00 and \$2.00 greater than for a depth of 14 inches. The lengths quoted for stringers are 24, 26, 28, 30, and 32 feet; all odd lengths to measure and take price of next even length above. The price of car sills, 3-corner heart grade, 16 inches wide, is 30 percent more than for a width of 8 inches, and that for a length of 40 feet is from about 27 to 35 percent more than for a length of 30 feet. The additional cost of surfacing on four sides is about \$1.00 per M feet.

Since the prices on different items like stringers, caps, guard rails, car sills, etc., vary from time to time, it is practically impossible to formulate any definite rule as to their relative cost which will apply at different times. This fact indicates the importance of the study of price lists by the designer.

For descriptions and illustrations of American woods used for structural timber, consult a book on The Principal Species of Wood: Their Characteristic Properties, by CHARLES HENRY SNOW, 1903. See also Bulletin No. 10 of the Division of Forestry, 1895, on Timber: An Elementary Discussion of the Characteristics and Properties of Wood, by FILIBERT ROTH; and Bulletin No. 41, 1903, on Seasoning of Timber, by HERMANN VON SCHRENK.

ART. 79. TIMBER TESTS.

The subject of this article is so extensive that the limits imposed upon this volume by its chief purpose make it impracticable even to give a summary of the methods employed in testing the conditions of growth and of use which modify the strength of timber, the general results which have been obtained by numerous experimenters, and of their relation to the design of structures. It would require more than eight pages to give the bibliography of original sources of information on the results of timber tests in America.

At the fifth annual convention of the Association of Railway Superintendents of Bridges and Buildings, held at New Orleans, La., in October, 1895, a report was adopted upon the strength of bridge and trestle timbers, which was prepared after careful study by a special committee, the chairman being WALTER G. BERG. This report was published in the Proceedings of the Association for 1895, was subsequently republished in a volume entitled American Railway Bridges and Buildings, by WALTER G. BERG, Chicago, 1898, pages 220 to 238, and 670 to 706, inclusive; and then republished separately in BERG's Complete Timber Test Record, Chicago, 1899. This committee studied all the literature on timber tests then available, and the report gives a complete list of references and abstracts of the results obtained. In the following paragraph are quoted some extracts from the introductory statement preceding the body of the report. The substance of some others is given in preceding chapters.

"The test data at hand and the summary criticisms of leading authorities seem to indicate the general correctness of the following conclusions: 1. Of all structural materials used for bridges and trestles, timber is the most variable as to the properties and strength of different pieces classed as belonging to the same species, hence impossible to establish close and reliable limits of

strength for each species. 2. The various names applied to one and the same species in different parts of the country lead to great confusion in classifying or applying results of tests. 3. Variations in strength are generally directly proportional to the density or weight of timber. 4. As a rule, a reduction of moisture is accompanied by an increase in strength; in other words, seasoned lumber is stronger than green lumber. 5. Structures should be in general designed for the strength of green or moderately seasoned lumber of average quality, and not for a high grade of well-seasoned material. 6. Age or use does not destroy the strength of timber, unless decay or season-checking takes place. 7. Timber, unlike materials of a more homogeneous nature, as iron or steel, has no well-defined limit of elasticity. [This statement is claimed to be incorrect; see Proceedings American Railway Engineering and Maintenance of Way Association, 1908, vol. 9, page 382.] As a rule, it can be strained very near to the breaking point without serious injury, which accounts for the continuous use of many timber structures with the material strained far beyond the usually accepted safe limits. On the other hand, sudden and frequently inexplicable failures of individual sticks at very low limits are liable to occur. 8. Knots, even when sound and tight, are one of the most objectionable features of timber, both for beams and struts. The full-size tests of every experimenter have demonstrated, not only that beams break at knots, but that invariably timber struts will fail at a knot or owing to the proximity of a knot, by reducing the effective area of the stick and causing curly and cross-grained fibers, thus exploding the old practical view that sound and tight knots are not detrimental to timber in compression. 9. Excepting in top logs of a tree or very small and young timber, the heart-wood is, as a rule, not as strong as the material farther away from the heart. This becomes more generally apparent, in practice, in large sticks with considerable heart-wood cut from old trees in which the heart has begun to

decay or been wind shaken. Beams cut from such material frequently season-check along middle of beam and fail by longitudinal shearing. 10. Top logs are not as strong as butt logs, provided the latter have sound timber. 11. The results of compression tests are more uniform and vary less for one species of timber than any other kind of test; hence, if only one kind of test can be made, it would seem that a compressive test will furnish the most reliable comparative results."

During the decade following this valuable report, whose recommendations have been widely adopted by designers, numerous experiments were made by the U. S. Forest Service, and by engineering laboratories and experiment stations. Nearly all of these later tests were made on full-size specimens of definite grades, according to the rules in practice for the selection of structural timber. In order to determine anew the proper working unit-stresses which should be adopted for the design of timber structures, the Committee on Wooden Bridges and Trestles of the American Railway Engineering and Maintenance of Way Association made a study of all the available results of tests, including some that had not been published, and in a preliminary report on the subject submitted a bibliography which is published in the Proceedings for 1907, vol. 8, pages 409–416, and which is probably fairly complete, provided the secondary references are all included.

The student's especial attention is called to a few of the titles contained in this bibliography. The first one is that of a paper presented by GAETANO LANZA to the International Engineering Congress at St. Louis in 1904, on Tests of Timber, giving a brief historical review of the subject and a statement of the general principles relating to methods and results. Another one is Circular 115 of the U. S. Forest Service, being the Second Progress Report on the Strength of Structural Timber, by W. KENDRICK HATT. This was issued Oct. 27, 1907, hence the Circular number could not be given in the bibliography mentioned above.

The following additional references should be consulted: Shearing Tests of Douglas Fir Wood with Oblique Loads (at 15, 30, and 45 degrees), in Tests of Metals, 1902, page 519. Circular 108 of the U. S. Forest Service, on The Strength of Wood as Influenced by Moisture, by HARRY DONALD TIEMANN, issued Aug. 26, 1907; Results of Recent Work of the Timber Tests by the Forest Service, U. S. Department of Agriculture, by W. KENDRICK HATT, in Appendix A to the Report on Wooden Bridges and Trestles, Proceedings American Railway Engineering and Maintenance of Way Association, 1907, vol. 8, pages 417–437; Discussion of Report on Wooden Bridges and Trestles in the Proceedings, 1908, vol. 9, pages 377–382; Report on Wooden Bridges and Trestles, in Proceedings, 1909, vol. 10, page 533; Abstract of Tests of Full-size Stringers made at the University of Illinois Engineering Experiment Station and Tests of Redwood at the University of California, in Appendix D of the preceding report.

The second reference given in the preceding paragraph includes a brief general review of the results of investigations on the effect of change of moisture condition on the strength; the effect of knots; the relative strength of commercial grades; a general table of strength of large timbers; on longitudinal shear; the effects of preservatives; and the time effect of stress on wood.

In item 11 of the quotation in the third paragraph of this article, reference is made to the importance of the simple test in compression parallel to the fiber. In Circular No. 18 of the U. S. Division of Forestry published in 1898, a relation was deduced between the compression endwise strength and the breaking load of a beam by S. T. NEELY. To test this relation, some special experiments were made at Cornell University under the direction of FILIBERT ROTH, by CLARENCE A. MARTIN, and GEORGE YOUNG, JR., the results of which are given in an

article in Railroad Gazette, vol. 35, page 185, March 13, 1903.

On account of the extensive use of bolts in connecting timbers, in which the bearing of the fibers upon the cylindrical surface of a bolt is an important element, there is an urgent need for experiments on the tensile strength of structural timber in a direction perpendicular to the fibers, or the resistance to splitting.

An interesting set of half-tone illustrations (17 plates) of fractures of American woods may be seen in Tests of Metals, 1883, page 649. They include fractures due to bending, compression parallel to the fibers, and indentation on the side of the fibers. Additional views for compression on the side of the fibers are published in Tests of Metals, 1896, page 388.

ART. 80. VARIATION IN STRENGTH OF TIMBER.

In determining the proper working unit-stress for any given species of wood, it is necessary to consider not only the average ultimate strength for the grade to which the unit-stress is to be applied in design, but the strength of the weaker specimens which are admitted under the grading rules. For those stresses in which the elastic limit can be determined this value is more important than the ultimate strength, and the lowest values of the elastic limit should be considered in a similar manner. It is desirable, therefore, to study the relative strength of the pieces which were used in the most extensive series of tests to which reference was made in the preceding article.

At the request of the Committee on Wooden Bridges and Trestles of the American Railway Engineering and Maintenance of Way Association, the U. S. Forest Service prepared, in 1908, an analysis of the full-size tests of structural timber, made during the previous six years in accordance with the instructions in Circular 38, published in 1906. A revised edition of the circular was issued March 31, 1909. The results of the tests

are classified to give the averages of groups of 10, 10, 30, 30, 10, and 10 percent, respectively, and show their relation from the lowest to the highest groups. The values are shown graphically on sheets 13 to 18 inclusive, in the Proceedings of the Association, 1909, vol. 10, page 533.

The relation of the average value for the lowest 10 percent group to that of the general average for each series, expressed as a percentage for each kind of timber, is given in the following paragraphs.

1. Modulus of Rupture. Douglas fir, green, 59.4; partially air-dry, 46.5; shortleaf pine, green, 73.3; loblolly pine, green, 60.7; partially air-dry, 61.8; Norway pine, green, 72.2; tamarack, green, 54.2; average, 61.2.

2. Modulus of Elasticity. Douglas fir, green, 69.5; partially air-dry, 71.0; shortleaf pine, green, 74.7; loblolly pine, green, 61.7; partially air-dry, 69.0; tamarack, green, 69.7; average, 69.3.

3. Shear Parallel to the Fibers. Douglas fir, partially air-dry, tangent to the annual rings, 56.8, and radially, 58.7; shortleaf pine, green, tangentially, 64.6, and radially, 62.1; Western hemlock, green, tangentially, 81.0, and radially, 79.8; Western larch, green, tangentially, 74.9, and radially, 64.5; average, 67.8.

4. Compression Perpendicular to the Fibers. Douglas fir, green, 66.3; shortleaf pine, green, 69.4; Western hemlock, green, 74.3; Western larch, green, 67.2; average, 69.3.

5. Compression Parallel to the Fibers. Douglas fir, green, 70.9; partially air-dry, 66.3; shortleaf pine, green, 70.6; Western hemlock, 86.2; Western larch, 80.7; average, 74.9.

6. General Averages. Douglas fir, 62.8; shortleaf pine, 69.1; loblolly pine, 63.3; Norway pine, 72.2; tamarack, 62.0; Western hemlock, 80.3; and Western larch, 71.8. Modulus of rupture, 61.2; modulus of elasticity, 69.3; shear parallel to the fibers, 67.8; compression perpendicular to the fibers, 69.3; and compression parallel to the fibers, 74.9.

The earlier tests (1891-96) by the Division of Forestry of the U. S. Department of Agriculture were made on a higher grade of carefully selected timber. From the summary of mechanical tests on thirty-two species of American woods, in Circular No. 15, the following percentages are computed for the relation of the lowest 10 percent group to the general average of each series.

1. Modulus of Rupture. Douglas fir, 50.7; longleaf pine, 80.7; shortleaf pine, 76.1; loblolly pine, 81.7; white pine, 63.3; bald cypress, 63.3; white cedar, 63.5; white oak, 58.5; average, 67.2.

2. Extreme Fiber Stress at the Apparent Elastic Limit. Douglas fir, 53.1; longleaf pine, 50.5; shortleaf pine, 66.6; loblolly pine, 65.2; white pine, 70.3; bald cypress, 63.6; white cedar, 69.0; white oak, 63.5; average, 62.7.

3. Compression Parallel to the Fibers. Douglas fir, 73.7; longleaf pine, 82.6; shortleaf pine, 81.3; loblolly pine, 83.2; white pine, 93.3; bald cypress, 71.6; white cedar, 84.6; white oak, 74.1; average, 80.5.

4. General Averages. Douglas fir, 59.2; longleaf pine, 71.3; shortleaf pine, 74.7; loblolly pine, 76.7; white pine, 75.6; bald cypress, 66.2; white cedar, 72.4; white oak, 65.4. Modulus of rupture, 67.2; extreme fiber stress at the apparent elastic limit, 62.7; and compression parallel to the fibers, 80.5.

The following percentages, in regard to the relative average strength for the lowest 10 percent groups, are computed from the published results of tests made in the engineering laboratory of the Massachusetts Institute of Technology, under the direction of GAETANO LANZA:

	YELLOW PINE	WHITE PINE	SPRUCE	WHITE OAK
Modulus of rupture	59.4	53.0	54.3	66.2
Modulus of elasticity	66.4	72.7	70.9	66.7
Longitudinal shear in beams . . .	50.8	——	51.7	——
Compression perpendicular to fibers	70.1	——	61.5	70.5

The lowest single ratio given above is 46.5 percent, being that for the modulus of rupture for Douglas fir which is partially air-dry. The value is 14 percent less than that of any other ratio given by the Forest Service, and is clearly abnormal, since the average modulus of rupture for the lowest 10 percent group is 17 percent less for the partially air-dry than for the green timber, while that of the next higher group is slightly greater, and the general average is over 5 percent greater.

The following statement to the Committee should be considered in comparing the average results for Douglas fir, transmitted in 1908, with those formerly published by the Forest Service: "The set of figures which was submitted by the Forest Service on June 6, 1908, was intended to show primarily the variation in strength from the average results shown in Circular 115. This set of results was obtained from material of a quality intermediate between merchantable and second, and the stresses are consequently somewhat lower than the average." The statement was also made that "the tests of the Forest Service show that, as a rule, in structural timber the added strength of the wood fiber due to drying out is offset by seasoning defects, such as checks, shakes, etc., although in some cases, where the timber has been carefully dried under shelter, an increase of 20 percent in strength has been observed. It has been found impossible to apply the moisture strength curves obtained from small specimens to structural sizes, since inherent defects in the material as well as seasoning defects have a very marked effect upon the strength of the material. Moreover, it has been found that the increase in strength due to seasoning varies with different grades, the greatest increase being in the select grades. The increase in strength due to seasoning varies in structural sizes from practically nothing to as much as 25 or 30 percent, depending upon species, grades, and several other factors. As suggested in Circular 115, one cannot safely assume a larger working stress for air-dry material than for green material in large sizes.

ART. 81. DEGREE OF SECURITY.

The working or allowable unit stress must equal but a fraction of the ultimate strength of the material. What is usually designated as the ultimate strength is obtained by a single application of the load which continues during comparatively few minutes. A much smaller load will ultimately break the material if applied a sufficient number of times, or if allowed to act continuously for a long time. Unless there is a reversal of stress no load will cause failure when the unit-stress due to it is below the elastic limit, no matter how many times the loading is repeated. It is therefore more rational to determine the working unit-stress with reference to the elastic limit than to the ultimate strength.

According to Circular 115 of the U. S. Forest Service, the relation of the elastic limit in flexure to the modulus of rupture, expressed as a percentage, is as follows: Douglas fir, 65.6; longleaf pine, 53.1; loblolly pine, 56.1; Norway pine, 64.1; tamarack, 61.6; Western hemlock, 64 2; average, 60.8. The corresponding relation of the elastic limit to the ultimate strength in compression parallel to the fiber is: Douglas fir, 79.1; longleaf pine, 72.5; loblolly pine, 69 3; Norway pine, 81.6; tamarack, 74.1; Western hemlock, 76.6; average, 75.5.

The following percentages are computed from the data given in Circular 15 of the Division of Forestry (1897) for the elastic limit in flexure: Douglas fir, 81.0; longleaf pine, 78 0; shortleaf pine, 78.2; loblolly pine, 82.0; white pine, 81.0; bald cypress, 83.5; white cedar, 92.1; white oak, 73.3; average, 81.1.

The state of knowledge concerning the strength of the material to be employed in construction must also be considered in determining the working unit-stress. The physical properties of some species of structural timber have been investigated

extensively, while comparatively few experiments have been made for other species. The references given in Art. 79 always indicate the number of tests in each case. The grade of material to be employed as determined by the relative cost of different grades at a given locality, should also be known if the design is to be made economically.

An important element in the relation between the proper working unit-stress and the elastic limit is the importance of the piece in the structure. In some cases the failure of a given piece endangers the entire structure, and may involve the loss of human life. In other cases the effect of failure may be remedied before any permanent injury occurs. The failure of a beam or column by flexure has more serious immediate effects than crushing a timber on the side of the fiber, which only shortens the life of the member, and increases the cost of maintenance.

Sometimes uncertainty exists in regard to the computed stresses. Errors in computation are always supposed to be eliminated by independent checking. The greatest uncertainty exists in trusses or framework having redundant members. While the method of least work may be employed in such cases, it is usually better to design structures so as to avoid redundant members wherever possible. As the joints are more or less fixed, the stresses differ somewhat from those computed under the usual assumption that the members are free to turn at the joints. When the stresses include the dynamic effect due to a moving load or the vibration of machinery, it is preferable to determine these effects separately, or to estimate them as closely as possible. Otherwise, the working unit-stress should be reduced according to the judgment of an experienced engineer.

Another element relates to the loads to be assumed in designing the structure. It is often difficult to obtain them with pre-

cision. The probable future increase of loads or of other changes in conditions during the life of the structure should be considered. In the case of roof trusses there may be a difference of opinion whether a heavy snow load and the maximum wind pressure should be regarded as acting in conjunction. Sometimes roof trusses in shops are designed to permit the hoisting of weights of a given magnitude, even though it may not be customary to use it occasionally for hoisting when the truss is first put into use. Stresses due to the erection of a truss should always be considered in designing its members.

Since the elastic limit for a piece subject to reversal of stress is materially lower than if there be no reversal, this condition is usually provided for in the specifications, sometimes by reducing the unit-stress, but preferably by determining additional section area in a different manner.

As timber becomes stronger due to seasoning when protected from the weather, and as some structures do not receive their maximum load until a considerable time has elapsed after construction, both of these conditions are to be taken into account in fixing the working unit-stresses to be employed. The more carefully all of the conditions are studied which have been mentioned in this article, including the inspection of the material to see whether it conforms to the specifications, the more economy may be secured in the design. Proper care in the protection of timber during the beginning of the seasoning process materially increases its value by preventing defects which otherwise reduce its strength, especially those which reduce its resistance to longitudinal shear and which permit the entrance of water and thus injure its durability.

In the report referred to in the second paragraph of Art. 79, the following statement occurs: "In addition to the ultimate breaking unit-stress the designer of a timber structure has to establish the safe allowable unit-stress for the species of timber

to be used. This will vary for each particular class of structures and individual conditions. The selection of the proper 'factor of safety' is largely a question of personal judgment and experience, and offers the best opportunity for the display of analytical and practical ability on the part of the designer. It is difficult to give specific rules. The following are some of the controlling questions to be considered: The class of structure, whether temporary or permanent, and the nature of the loading, whether dead or live. If live, then whether the application of the load is accompanied by severe dynamic shocks and pounding of the structure. Whether the assumed loading for calculations is the absolute maximum rarely to be applied in practice, or a possibility that may frequently take place. Prolonged heavy, steady loading, and also alternate tensile and compressive stresses in the same piece, will call for lower averages. Information as to whether the assumed breaking stresses are based on full-size or small-size tests, or only on interpolated values averaged from tests of similar species of timber, is valuable, in order to attribute the proper degree of importance to recommended values. The class of timber to be used, and its condition and quality. Finally, the particular kind of strain the stick is to be subjected to, and its position in the structure with regard to its importance and the possible damage that might be caused by its failure."

It is an important matter to decide whether the working unit-stress shall be determined by giving considerable weight to the minimum strength of a given species of timber or to adopt a higher unit-stress and to replace promptly any stick that may show signs of weakness, careful inspection being constantly maintained. A valuable contribution to this discussion may be found in an article on Standard Plans for Pile and Timber Trestle Bridges, Santa Fe Railway System, by A. F. ROBINSON, in Proceedings American Railway Engineering and Maintenance of Way Association, 1905, vol. 6, page 43; and also in an

article on What is the Life of an Iron Railroad Bridge; by J. E. GREINER, in Transactions American Society of Civil Engineers, vol. 34, pages 301 and 302.

Reference should also be made to Working Stresses and Working Factors of Safety in Structures, by A. N. TALBOT, in Engineering News, vol. 58, page 33, July 11, 1907; to an editorial on Elastic Limit or Yield-point as Guide for Working Stresses in Engineering News, vol. 62, page 150, Aug. 5, 1909; and to an editorial on Working Stresses for Timber, in Engineering Record, vol. 60, page 225, Aug. 28, 1909.

ART. 82. WORKING UNIT-STRESSES.

The accompanying table gives the working unit-stresses for structural timber adopted by the American Railway Engineering and Maintenance of Way Association. As stated in the note at the head of the table, the unit-stresses primarily are intended for railroad bridges and trestles, but the tabulated values may be used for other structures after increasing them by given percentages.

These unit-stresses are the result of an extended study of all the full-size tests of structural timber available, as well as of the unit-stresses which have been in use in designing wooden bridges and trestles and have been demonstrated to be safe by extensive experience. The original sources of information for different kinds of timber are given in Appendix B of the report of the Committee in Proceedings American Railway Engineering and Maintenance of Way Association, 1909, vol. 10, page 533, the table being given on page 564. As indicated in the footnote the values are based on a green condition of the timber, but in a few cases where no data for green timber were available, those for partially air-dry timber were inserted.

The table contains no working unit-stress for pure tension. Wood has a greater resistance to tension than to any other kind

UNIT-STRESSES FOR STRUCTURAL TIMBER EXPRESSED IN POUNDS PER SQUARE INCH.

Adopted by the American Railway Engineering and Maintenance of Way Association upon the Recommendation of the Committee on Wooden Bridges and Trestles.

Note. — The working unit-stresses given in this table are intended for railroad bridges and trestles. For highway bridges and trestles the unit-stresses may be increased 25 percent. For buildings and similar structures, in which the timber is protected from the weather and practically free from impact, the unit-stresses may be increased 50 percent. To compute the deflection of a beam under long-continued loading instead of that when the load is first applied, only 50 percent of the corresponding modulus of elasticity given in the table is to be employed.

KIND OF TIMBER.	BENDING.			SHEARING.				COMPRESSION.						
	Extreme Fiber Stress.		Modulus of Elasticity.	Parallel to the Grain.		Longitudinal Shear in Beams.		Perpendicular to the Grain.		Parallel to the Grain.		For Columns under 15 Diams. Working Stress.	Formulas for Working Stress in Long Columns over 15 Diameters.	Ratio of Length of Stringer to Depth.
	Average Ultimate.	Working Stress.	Average.	Average Ultimate.	Working Stress.	Average Ultimate.	Working Stress.	Elastic Limit.	Working Stress.	Average Ultimate.	Working Stress.			
Douglas Fir	6100	1200	1 510 000	690	170	270	110	630	310	3600	1200	900	$1200(1 - l/60d)$	10
Longleaf Pine	6500	1300	1 610 000	720	180	300	120	520	260	3800	1300	980	$1300(1 - l/60d)$	10
Shortleaf Pine	5600	1100	1 480 000	710	170	330	130	340	170	3400	1100	830	$1100(1 - l/60d)$	10
White Pine	4400	900	1 130 000	400	100	183	70	290	150	3000	1000	750	$1000(1 - l/60d)$	10
Spruce	4800	1000	1 310 000	600	150	170	70	370	180	3200	1100	830	$1100(1 - l/60d)$	
Norway Pine	4200	800	1 190 000	590*	130	250	100	—	150	2600*	800	600	$800(1 - l/60d)$	
Tamarack	4600	900	1 220 000	670	170	261	100	—	220	3200*	1000	750	$1000(1 - l/60d)$	
Western Hemlock	5800	1100	1 480 000	630	160	270*	100	440	220	3500	1200	900	$1200(1 - l/60d)$	
Redwood	5000	900	800 000	300	80	—	—	400	150	3300	[illegible]	680	$900(1 - l/60d)$	
Bald Cypress	4800	900	1 150 000	500	120	—	—	340	170	3900	1100	830	$1100(1 - l/60d)$	
Red Cedar	4200	800	860 000	—	—	—	—	470	230	2800	900	680	$900(1 - l/60d)$	
White Oak	5700	1100	1 150 000	840	210	270	110	920	450	3500	1300	980	$1300(1 - l/60d)$	12

These unit-stresses are for a green condition of timber and are to be used without increasing the live load stresses for impact.

* Partially air-dry.

l = Length in inches.
d = Least side in inches.

of stress, and it is found to be difficult to break it in a true tensile test. As there is more or less cross-grain, it is advisable to use the same unit-stress in designing tensile members as for bending. The formulas for long columns are straight-line formulas and it will facilitate their use to plot them on a sheet of cross-section paper.

ART. 83. BUILDING CODES.

For cities of a given class, laws are enacted by the states and ordinances by the cities to regulate the construction, maintenance, and inspection of buildings. These include loads and allowable unit-stresses as well as many other conditions which are to be fulfilled. The accompanying tables contain the permissible unit-stresses for eight of the principal cities.

It will be noticed that the species of wood included in the lists are few in number. No distinction is made between longleaf and shortleaf yellow pine. The unit-stresses are very low, being apparently adapted for a low grade of lumber. They represent the lowest limit which is allowed to pass the bureau of inspec-

SAN FRANCISCO, 1907.

KIND OF STRESS.	WHITE PINE AND SPRUCE.	DOUGLAS FIR.	REDWOOD.
Bending	700	1100	750
Tension	700	1200	700
Shearing parallel to grain	100	150	100
Compression perpendicular to grain	200	300	200
Compression parallel to grain	1100	1600	1000
Compression of columns under 15 diameters	700	1200	800
Tension across the grain	50	200	40

tion. It is assumed that many owners will voluntarily use a higher quality of material in order to secure better buildings. In the cities of Washington, D.C., New York, Philadelphia, and Toronto the code also includes shear across the fiber. It is very doubtful whether wood ever fails under that stress, and hence the values are not reprinted in this article.

PERMISSIBLE UNIT-STRESSES IN TIMBER.

(Expressed in Pounds per Square Inch.)

KIND OF STRESS.	KIND OF TIMBER.	BOSTON, 1907.	CHICAGO, 1900.	DISTRICT COLUMBIA, 1902.	NEW YORK, 1899.	PHILADELPHIA, 1899.	TORONTO. 1907.
Bending	Yellow pine	1500	1250	1200	1200	1600	1440
	White pine	1000	750	800	800		900
	Spruce	1000	750	800	800	1100	900
	Oak	1000	1000	1000	1000		1080
	Locust			1200	1200		
	Hemlock				600	900	810
	Chestnut			800	800		
Tension	Yellow pine			1200	1200	1200	1440
	White pine			800	800		900
	Spruce			800	800	800	900
	Oak			1000	1000		1080
	Hemlock				600	600	810
Shearing parallel to the grain	Yellow pine	100	100	70	70	67	70
	White pine	80	80	40	40		42
	Spruce	80	80	50	50	50	42
	Oak	150	150	100	100		70
	Locust			100	100		
	Hemlock				40	42	42
Compression perpendicular to the grain	Yellow pine	500	250	600	600	550	500
	White pine	250	150	400	400		300
	Spruce	250	150	400	400	300	300
	Oak	600	250	800	800		600
	Locust			1000	1000		
	Hemlock				500	250	300
	Chestnut			1000	1000		
Compression parallel to the grain	Yellow pine			1000	1000	750	1100
	White pine			800	800		900
	Spruce			800	800	500	900
	Oak			900	900		1100
	Locust			1200	1200		
	Hemlock				500	350	810
	Chestnut			500	500		

Buffalo, 1896: Bending; yellow pine, 1800; white pine, 1080; oak, 1350; hemlock, 1080.

INDEX.

s book may be kent